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DESIGN OF LARGE PRESSURE CONDUITS IN ROCK[a]

F. W. Patterson,[1] R. L. Clinch,[2] and I. W. McCaig[3]
(Proc. Paper 1457)

ABSTRACT

The design of pressure conduits in rock is influenced by the available rock cover and the characteristics of the rock. When steel liners are installed in pressure conduits, there arise the problems of proportioning of internal pressure between steel and rock and of the design of the steel against external pressure.

The paper presents a review of the theoretical principles and design assumptions which influence the design of pressure conduits in rock and describes the design, fabrication and construction of pressure conduits for three power developments in the Province of Quebec.

INTRODUCTION

Until the last decade, and with few exceptions, Canadian hydro-electric developments were in the low-to-medium head range, advantage being taken of the existence of economic sites close to industrial centres, on rivers having a high annual runoff. In recent years, however, a number of major developments have been constructed both for the electro-metallurgical industry and for public utilities at sites more remote from the load centres and utilizing heads between 387 feet and 2,600 feet. The trend towards the use of units of ever-increasing capacity has created a number of interesting problems in the design of pressure conduits and, in particular, of steel-lined conduits in rock.

For example, the Spray development completed in 1951, and operating

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1. Chief Project Engr., H. G. Acres & Co., Ltd., Niagara Falls, Canada.
2. H. G. Acres & Co., Ltd., Niagara Falls, Canada.
3. H. G. Acres & Co., Ltd., Niagara Falls, Canada.

under a gross head of 905 feet, has a steel liner 7 feet 6 inches in diameter at the downstream end of the pressure conduit.(1) Another recent development of note in relation to conduit design and, for that matter, in a number of other respects, is the Kemano development in which the eight Pelton units presently being installed are supplied by two steel-lined pressure conduits. These steel liners are 11 feet in diameter and are subjected to a static head of 2,600 feet.(2)

It is proposed to present, in this paper, the design of pressure conduits for three hydro-electric developments in the Province of Quebec and to describe in some detail the fabrication and construction of the pressure conduits for one of these plants.

Referring to Figure 1, it may be seen that two of the developments are located on the Bersimis River and the third on the Peribonka River. The uppermost development on the Bersimis River, namely, Bersimis No. 1, is already in operation, while the Bersimis No. 2 development and the Chute-des-Passes development on the Peribonka River are both under construction at the present time.

In each of these three developments, advantage has been taken of the economy which results from the use of turbines of large capacity and hence the pressure conduits are of considerable diameter. The static heads on the steel-lined sections of conduit vary from 387 feet at Bersimis No. 2 to 875 feet at Bersimis No. 1. Other pertinent information concerning these developments is given in Table I.

At the outset it should be stated that the three sites are located in the Pre-Cambrian shield where tunnelling conditions are generally good. Taking advantage of the geology and topography of the region, therefore, the conduits have been driven at depths which provide good rock cover and, as a result, only relatively short lengths of steel liner have been required adjacent to the power houses.

Before dealing with the subject as it relates to these three projects, it will be useful to review the theoretical principles and design assumptions which apply generally to the design of pressure conduits in rock. In discussing the application of these principles to the specific projects mentioned above, an attempt is made to show how the design has been influenced by construction requirements.

Rock Stresses Before and After Tunnelling

The magnitude of stresses which exist in any mass of rock through which a pressure conduit has to be constructed is an influencing factor in the design of that conduit, and hence an estimation or determination of these stresses is of importance.

Terzaghi and Richart,(3) in their theoretical treatment of rock stresses around cavities, have shown that, at depth z below the horizontal surface of a mass of rock with unit weight w, the vertical stress is given by the equation

$$\sigma_{v_o} = zw$$

If the rock were free to expand in all horizontal directions, no other stress than this would exist in the rock. However, the lateral expansion of the rock is prevented by the resistance of adjacent rock masses and horizontal

TABLE I

Item	Bersimis No. 1	Bersimis No. 2	Chute-des-Passes
Installed capacity ...	1,200,000 hp	855,000 hp	1,000,000 hp
Number of units	8	5	5
Static head	875	387	640
Concrete-lined supply tunnel - equivalent diameter ..	31'-0"	38'-0"	34'-4"
Concrete-lined pressure conduits - diameter	12'-0"	17'-0"	None
Steel-lined pressure conduits:			
Diameter	10'-0" to 7'-9"	17'-0" to 12'-0"	15'-0" to 11'-0"
Plate thickness	1-1/16" to 2-1/16"	3/4" to 1-9/16"	1-1/2" to 2"
Design head	1240'	510'	920'
Steel specification.	A201 - Grade B Firebox quality	A201 - Grade B Firebox quality	A201 - Grade B Firebox quality, fine grain practice

stresses are induced. These stresses may be defined by the equation

$$\sigma_{h_o} = Nzw$$

where N is a dimensionless factor which depends on the geological history of the rock. For rock deposited in layers in geologically undisturbed regions, Terzaghi and Richart give values of N between 0.11 and 0.43.

Measurements made by McHenry and Olsen of the U.S.B.R. at the Gorge penstock extension tunnel located beneath a ridge have indicated a value of N approximately equal to 0.6, while measurements in a tunnel at Hoover Dam located beneath a steep-sided valley showed that preloading in past geological time had caused the value of N to rise to approximately 3. On the other hand, European investigators(4) suggest that rock at depth can be considered as a plastic material and N can be taken as approximately equal to unity.

The wide range of measured values for the ratio between horizontal and vertical stresses in rock and the divergence even in the theoretical approach of investigators is a good indication that estimates of this stress ratio for any particular development are not likely to be too reliable and that, if possible, field tests should be carried out. Unfortunately, however, most field tests which have been suggested involve measurements being made inside an exploratory tunnel or inside the actual pressure conduit, a process which is often impracticable due to time limitations. For example, in the three developments covered in this paper it was necessary to order steel liners before excavation of the tunnels had started. Furthermore, these test methods can be quite inaccurate unless very light charges are used in excavating the test section of tunnel to avoid shattering the rock. It is suggested that it might well be possible to develop a method whereby horizontal to vertical stress ratios at any given location in a mass of rock could be measured in exploratory drill holes.

When a tunnel is driven through rock, the initial stresses in the rock normal to the periphery of the tunnel are reduced to zero. Consequently, there is a redistribution of stress around the opening. If the horizontal and vertical stresses are initially equal, the tangential stresses adjacent to the periphery of a circular tunnel will rise, according to elastic theory, to approximately twice the initial value of wz.(5) However, if the initial horizontal stress is only one-third of the vertical stress, the tangential stress on the vertical walls will be 2.67 wz while on the soffit and invert it will be zero. The stress distribution for these two cases is shown on Figures 2(a) and 2(b).

Rock Stresses Due to Water Pressure in a Conduit

When an internal pressure is applied to a circular tunnel whose depth below the rock surface is several times the tunnel diameter,(6) radial compressive stresses and tangential tensile stresses are induced at the inside face, equal in magnitude to the internal pressure. These radial and tangential stresses in the rock diminish in proportion to the square of the distance from the tunnel centreline as shown on Figure 3.

A comparison of Figures 2(a) and 3 will show that if the horizontal stress in the rock is initially equal to the vertical stress wz, water pressure in the tunnel can rise as high as p = wz without the compressive stress in the rock being reduced to below the stress which existed prior to tunnel excavation.

Under such ideal conditions, the factor of safety against tension stress occurring will be p/2wz and, if w is taken as 156 pounds per cubic foot, the cover can be as low as 20 per cent of the head in the tunnel before the safety factor is reduced to unity.

However, when the initial horizontal stresses in the rock are less than the vertical stresses (N less than unity), tension stresses in the rock, with consequent risk of cracks and leakage at the soffit, will occur at a pressure much less than 2wz. For N equal to 1/3, cracks will start as soon as pressure is applied and will penetrate deep into the rock when the pressure p is greater than 1/3 wz.

It is considered that the theoretical approach given above, although assuming ideal conditions, is useful in illustrating the phenomenon which can occur when a tunnel in rock is subjected to an internal pressure. Unfortunately, however, very few quantitative values for rock properties, and in particular for the value N for rock in a given locality, are available, and it is usually necessary to decide a minimum cover required over any given pressure conduit from experience in similar rock formations and from a study of the geological history associated with the rock formation in question.

Because of the uncertainty which exists with regard to the rock properties, it is sometimes considered that rock cover over a pressure conduit which is not steel lined should never be less than 50 per cent of the maximum internal pressure head which can be experienced. With this amount of cover, and using the theoretical approach given above, cracks in the rock above the soffit of the tunnel will not occur if the ratio of the initial horizontal stress to the initial vertical stress in the rock is greater than 0.46. If this ratio is less than 0.46, cracks and consequent leakage can occur; but with a unit weight of 156 pounds per cubic foot there still exists a factor of safety of 1.25 against heaving of the full depth of overlying rock.

Unlined and Concrete-Lined Pressure Conduits

Since the elastic properties of a concrete lining in a pressure conduit are approximately the same as those of the surrounding rock, a concrete-lined tunnel will behave in a similar fashion to an unlined tunnel in resisting internal water pressure and the theoretical consideration given in the preceding section will apply, provided the concrete lining is in intimate contact with the rock.

The need, or otherwise, of a concrete lining is often determined by economic rather than structural considerations in that a concrete lining reduces hydraulic friction loss. In some special cases, however, a concrete lining can be essential; for example, in rocks in which the ratio of the initial horizontal to vertical stress is low, tension can easily occur at the soffit in an unlined tunnel when water pressure is applied. Since rock at the soffit of a tunnel is often shattered and weakened by the explosives used in driving the tunnel, there is a strong possibility that when the internal pressure is applied rockfalls from the roof of the tunnel will occur. In such cases concrete may be required to prevent a partial blockage of the tunnel which could result in an excessive increase in friction loss.

When a concrete lining is employed, a number of additional design problems arise in that, on dewatering the tunnel, it is possible for the lining to be placed in compression due to external water pressures. Furthermore, in

some cases, considerable stress can be developed in the lining due to plastic flow occurring in the surrounding rock after the lining has been placed. Limestones, sandstones and shales appear to be susceptible to this phenomenon.

With modern concrete placing techniques, it is possible to construct concrete linings having an average thickness as small as 9 inches, and collapse of such a thin lining could be caused by either of the phenomena mentioned above. Such a possibility must, therefore, always be investigated in the design of the concrete lining of pressure conduits, and methods of calculating the stresses which could occur are available.(7)

Steel-Lined Pressure Conduits

There are a number of factors which can influence the decision as to whether a steel lining is required in any given section of a pressure conduit, the most important, perhaps, being the need to prevent excessive leakage from the conduit at points where the rock cover might be insufficient to prevent rock heave.

The arguments concerning the required vertical cover are given in Section 3, but it is often the case that, as a pressure conduit approaches the power house, the horizontal distance of the pressure conduit to the surface of the ground or to the power house excavation is small and the possibility of horizontal rock movement in such cases has to be investigated.

In badly jointed or faulted rock, it is possible that, although rock cover is sufficient to prevent rock movement, leakage from the conduit may be excessive or may be sufficient to cause movement or sliding of overburden. In such cases it is necessary to extend the steel lining beyond the point to which it would normally be required to prevent possible rock movement. Steel linings are therefore used in pressure conduits not only for structural strength but also to act as a watertight membrane.

The thickness of a steel lining is small in relation to the diameter of the pressure conduit and the design is very often determined by a consideration of the external pressure which can be exerted by ground water when the conduit is dewatered, rather than by internal hydrostatic pressure. This is especially true for conduits subject to maximum internal pressure heads less than 1,000 feet, as will become apparent from the discussion which follows.

Design Against External Pressure

The collapse of a thin steel liner due to buckling under external pressure was studied by Amstutz(8,9) and Vaughan(10) who have both produced design curves having the same general trend. However, due to a variation in assumptions, there is a substantial difference between the numerical values of critical external pressure obtained by the two methods for any given set of conditions, and for this reason it is necessary to take a more empirical approach to the problem.

The published figures of external pressures which have been applied to existing conduits either by grouting or during dewatering have been plotted against the ratio of conduit diameter to steel liner thickness on Figure 4, together with the curve derived from the revised theory of Amstutz and assuming that a gap of three ten-thousandths of the conduit radius exists between the steel liner and the surrounding concrete. No effective stiffening

against external pressure was provided in any of the steel liners used in this plot. The circles indicate that the conduit was grouted or dewatered without buckling, whereas the crosses indicate that buckling occurred at some point along the length of the liner during one of these operations. An arrow indicates that there is doubt as to the precise position at which the point should be plotted.

It can be seen that most of the failures have occurred at pressures greater than the critical external pressure as given by the curve shown on Figure 4, whereas most of the steel liners which were subjected to less than this critical external pressure did not buckle. It is suggested that publication of any relevant data additional to that shown on Figure 4 will be of material help in the design of steel-lined pressure conduits.

One of the assumptions used in deriving the curve given on Figure 4 concerns the thickness of the gap which will be present between the steel liner and the surrounding concrete. The smaller the gap, the greater the critical external pressure. The thickness of the gap depends on site conditions and the construction techniques employed which, in turn, may be dictated to some extent by the construction schedule. All these factors, therefore, must be taken into account in the design of steel liners to resist external pressure.

The temperature of a steel liner will be raised by heat generated during the setting of the surrounding concrete, and a gap will therefore form when the conduit is filled with water at a lower temperature than the steel. For example, if the temperature drop on filling a 14-foot diameter steel-lined conduit with water is 25 degrees F, a gap of approximately thirteen-thousandths of an inch will form.

When the conduit is filled with water under high pressure, the shrinkage gap will be closed and pressure exerted on the surrounding concrete and rock. Radial cracks will form in the concrete and rock, and tangential cracks or joints in the shattered rock adjacent to the concrete will close. If, then, the conduit is dewatered, the resulting gap will be greater than the original shrinking gap.

The extent of the probable rock deformation for a given conduit is difficult to assess. However, test results on a number of European conduits suggest that, even if the surrounding rock is carefully grouted, the shrinkage gap can be increased by as much as twenty-thousandths of an inch, giving a total gap in the example of the 14-foot diameter conduit amounting to thirty-three thousandths of an inch.

There are at least three methods by which the required thickness of a steel liner to resist external pressure can be reduced. Drainage is an obvious solution and has been employed in several recent developments. However, a system of piping which is required to ensure that external pressure is relieved over the entire surface of the steel liner can become quite cumbersome and can also require an increased length of time for construction. There is, furthermore, at least one case on record in which failure of the steel liner has occurred(4) due to excessive stresses being set up adjacent to a drainage tunnel running parallel to a steel conduit.

The second method, whereby the necessary thickness of a steel liner subject to external pressure can be reduced, is to limit, by various means, the size of the gap which will exist between the steel liner and the surrounding concrete. Even when construction is to a fast schedule, it is usually possible to limit deformation of the surrounding rock by grouting and thus reduce the ultimate gap size. In addition, and where construction and operational

requirements allow, it is possible to reduce the gap still further by dewatering the conduit some time after it has first been subjected to internal hydrostatic pressure and regrouting outside the liner. Such a procedure will, of course, not be possible where relief drains are incorporated and in operation.

The third method, often employed by some designers to reduce the thickness of the steel liner subject to external pressure, is to weld to the outside surface of the liner a large number of steel anchors which will support the steel liner after concrete is placed between the liner and the surrounding rock.

So far, no mention has been made of the design head used in computing the external load for a pressure conduit steel liner. The magnitude of the external pressure which may be experienced on dewatering the conduit is closely related to site conditions and, hence, generalizations are difficult. At first sight it may be considered that the external pressure could be as high as the internal pressure, and such can be the case if a truly impervious barrier exists in the rock surrounding the pressure conduit. However, such an occurrence is very unlikely. It is generally more reasonable to assume that a small amount of leakage will occur to ground surface from the adjacent section of unlined or concrete-lined pressure conduit, and from the rock surrounding the steel liner, and that the maximum external pressure head that can exist outside the steel lining will not exceed the depth of cover over the steel lining.

Design Against Internal Pressure

As indicated at the beginning of this section, the internal pressure in steel-lined conduits is an important factor in the design of the steel liner for high head conduits and for low head conduits with small cover or adjacent to a power house excavation.

Formulae to determine the portion of the total load which will be taken by the surrounding rock and by a steel liner subject to internal pressure have been proposed by Jaeger,[4] Vaughan[10] and Bleifuss,[11] and the methods set out in Appendix I, and used by the authors, offer nothing new in this field. From an examination of the formulae given in Appendix I, it can be seen that the thickness of the steel liner has little effect on the stress which will occur in the steel liner, provided the liner thickness is less than conduit diameter divided by 200.

Considering now the possible variation in the elastic properties of rock surrounding a steel liner, it has been found that for Pre-Cambrian Rocks of the type found in the Canadian shield, Young's modulus ranges between one million psi for schists to six million psi for massive granodiorites. Using the example of a 14-foot diameter pressure conduit with a steel liner of one inch thickness or less, subject to internal pressure heads of 500, 1,000 or 2,000 feet, the stresses in the steel liner will be as given in the following table:

Design Head Ft	Stress in Steel Liner		
	E for Rock $= 10^6$	E for Rock $= 3 \times 10^6$	E for Rock $= 6 \times 10^6$
500	8,670	2,800	1,445
1,000	17,340	5,800	2,890
2,000	24,680	11,700	5,780

The figures of stress in a steel liner given in the above table are calculated on the assumption that no gap exists between the steel liner and the surrounding concrete, and hence they are lower than would occur in practice. However, they do serve to indicate the variations in the stress which would occur in steel liners surrounded by Pre-Cambrian rock having a wide range of elastic properties.

Assuming an average quality rock with Young's modulus of 3×10^6, calculations have been made to illustrate the variation in steel stress for the same 14-foot diameter tunnel but with a gap existing between the steel liner and the surrounding concrete. The results of these calculations are given on Figure 5, from which it can be seen that, for average rock quality and for all reasonable sizes of gap, stresses in a steel liner subject to internal pressure heads less than 1,000 feet do not exceed 20,000 psi.

The effect of rock cover on steel liner stress is illustrated by Figure 6. These values have been computed assuming average quality rock and zero gap between the steel liner and the surrounding concrete. It is interesting to note that the rock cover can be reduced to fairly low values before a material increase in the stress occurs in the steel liner. However, when the vertical cover is less than, say, four times the diameter of the pressure conduit or when the pressure conduit approaches a rock face, as, for example, is the case at the upstream wall of an underground power house, it is desirable to design for a greater safety factor than might be required for other sections of steel-lined pressure conduit.

In northern climates, the problem of brittle failure of steel must also be considered. Charpy tests on small specimens of rimmed and semi-killed steel show that a ductility transition of the steel takes place at a temperature around 40 degrees F. Since water temperatures at some power sites can be as low as 35 degrees F, it is advisable to specify fully killed steel to ensure a ductility transition temperature below the probable minimum temperature of the water. Even then, it may be considered desirable to place an arbitrary limit on the stress which could occur in the liner.

Where a limitation on steel stress due to internal pressure is adopted and where the design against external pressure is as previously described, there will exist a definite relationship between the tensile stress in the steel liner without rock support, the head in the pressure conduit and the design external pressure, expressed as a percentage of the design internal pressure, all as shown on Figure 7. For example, in a development where the head in the pressure conduit is 1,000 feet and where the tensile stress in the steel liner, due to internal pressure and without rock support, is limited to the yield

stress of, say, 30,000 pounds per square inch, the liner will be capable of withstanding an external pressure equal to 50 per cent of the internal pressure, i.e., 500 feet head. If relief drainage is not provided and if it is assumed that the external pressure head can be as high as the depth of rock cover over the conduit, then, on this basis, external pressure generally will be the governing design criteria if the maximum internal pressure head is less than 1,000 feet. It should be noted that the examples given on Figure 7 have been plotted in relation to their static heads since information as to the maximum internal pressure is not available in many cases.

Bersimis No. 1 Pressure Conduits(12)

In the preceding sections of this paper, an attempt has been made to set out the general principles involved in the design of pressure conduits. It is now proposed to describe the application of these principles to the design of three recent developments, and in the case of the Bersimis No. 1 development, which has been completed, to give a description of the fabrication and construction methods used.

Referring to Figure 8, it is seen that the 31-foot diameter concrete-lined supply tunnel at Bersimis No. 1 is 40,294 feet long. The supply tunnel terminates in a manifold from which eight concrete-lined tunnels dip at an angle of 50 degrees to the level of the underground power house. The rock in the area is identified as paragneisses and massive granites. The rock structure was generally very favourable for tunnelling. Although some major faults were intersected in the supply tunnel, no faults were encountered in the pressure conduits located downstream from the manifold.

The schedule of construction for the Bersimis No. 1 development did not allow of any field determination of the horizontal to vertical stress ratio in the rock at critical points along the length of the pressure conduit, and the minimum cover used in design was therefore determined from a consideration of the geological history of the area and the nature of the rock at the site. The Pre-Cambrian rocks of the Canadian shield have been subjected to considerable preloading by an ice sheet, and although subsequent erosion has occurred the value of N for the rock is unlikely to be less than 0.46, especially when it is considered that the intersecting sets of joint planes which exist in the rock are generally steeply inclined. If the value of N is equal to or greater than 0.46, cracking of the rock will not occur with a cover equal to 50 per cent of the maximum internal pressure, and this amount of cover was therefore used in design as the minimum requirement. As pointed out in Section 3, this amount of cover will also give a factor of safety of at least 1.25 against rock heave.

The supply tunnel and steeply sloping penstocks were arranged to provide a minimum of 50 per cent rock cover throughout their length. In addition, checks were made to ensure safety against rock heave above the steeply sloping penstocks along planes other than the vertical.

The supply tunnel was excavated at six faces using three adits, and the eight branch conduits were excavated from the power house by driving pilot shafts upward and then driving downward at the full bore. All sections of the supply tunnel, manifold, and branch conduits, except those adjacent to the power house, were concrete lined. The concrete linings were designed to withstand the external pressure due to ground water with the conduits

dewatered. The tunnel bore provided for a minimum and average thickness of concrete lining of 12 inches and 18 inches respectively. However, due to overbreak, the over-all average thickness was approximately 20 inches.

Grouting was carried out at the tunnel crown throughout the full length of the supply tunnel and concrete-lined sections of penstocks to close the gap between concrete and rock, and was performed in two stages to a pressure of 100 pounds per square inch. A program of high pressure grouting was also carried out in all areas where consolidation of the rock appeared necessary, or where water was encountered during the tunnelling operation. Complete geological mapping of the rock in the tunnels made it possible to lay out the grout patterns for most of the grouting after concreting had been completed.

Rock cover equal to 50 per cent of the maximum internal pressure could not be maintained in the final 328 feet of near horizontal penstocks adjacent to the underground power house and steel liners were therefore provided (Figure 9). Due to the proximity of these liners to the power house, it was desirable to apply a liberal factor of safety to the design. The maximum internal pressure at the steel-lined section amounted to 1240 feet, and since it was decided to limit the stress due to internal pressure to the yield stress assuming no rock support, internal pressure was the controlling factor in design at the upstream end. The downstream ends of the liners were designed to withstand the full internal pressure at a stress no higher than 15,000 psi. Stresses at intermediate points were set to give a reasonable transition. The plate thickness varied from 1-1/16 inches upstream to 2-1/16 inches at the downstream ends, and the diameter was reduced from 10 feet at the upstream ends to 7 feet 9 inches at the downstream ends to give maximum economy.

Immediately upstream from the power house a 24-foot section of each liner was coated with 1/8-inch of bitumen so that the thrust on the liners, as a result of closing the power house rotary valves, would not be transmitted to the power house wall.

The steel used in fabricating the pressure conduit steel liners was to ASTM Specification A201, grade B, firebox quality, which combines controlled high quality with low carbon and manganese content. The low carbon and manganese content associated with an A201 steel gives good welding characteristics and, since plate thicknesses were as great as 2-1/16 inches, this was an important consideration. In this respect, grade B was used as the slightly higher stress permitted with this grade kept the maximum steel thickness to 2-1/16 inches. By using a firebox quality steel, slag intrusions and laminations in the plate, detectable by the uniformity test, were kept to a minimum.

Fabrication of the steel liners was carried out in Montreal, from whence they were shipped by boat to Forestville on the north shore of the St. Lawrence River and thence by road to the site. Clearances on the route were not a problem, and ample crane capacity was available at the point of manufacture, at the wharf, and at the power house. It was therefore possible to ship the liners in lengths of approximately 50 feet, this length being determined by the problems of handling the liners inside the underground power house and sliding them into the penstock tunnels.

The Unionmelt process was used in making both the circumferential and longitudinal joints of each 50-foot section of steel liner. Two passes were required for plates less than 1-1/4 inches in thickness, one pass being made from each side of the joint. For plates of between 1-5/16 inches and 1-19/32

inches in thickness, four passes were required; for plates of 1-5/8 inches thickness and greater, five passes were used. The quality of all shop welding was checked by means of a test coupon welded to the end of all longitudinal joints, the coupon being cut off after welding of the joint, and then tested in accordance with the ASME Boiler and Pressure Vessel Code. In addition to the normal requirements laid down by the code, a sample of weld metal was cut from the coupon and tested for impact resistance using the Charpy V-notch test. It was required that the weld material should have an impact resistance of 15 foot-pounds in this test when conducted at a temperature of 34 degrees F. All shop welded joints were radiographed using X rays and were stress relieved in a furnace before shipping to the site.

On delivery at the power house the 50-foot long sections of steel liner were placed on trolleys and hauled into the penstock tunnels, this operation being shown in the photograph (Figure 10). Having been hauled to the correct location in the tunnel, each section of liner was jacked from the trolleys and accurately set on concrete pads (Figure 11) before welding to the preceding section of pipe. The circumferential field joints were made by hand, the top half being welded from the outside and the bottom half from the inside of the liner to give the maximum of downhand welding. All field joints were preheated to 300 degrees F and no welding was carried out at air temperatures below 50 degrees F. All field joints were low temperature stress relieved using the Linde method and radiographed throughout using gamma rays. The low temperature stress relieving equipment is shown in Figure 12.

Since the penstocks were excavated to the diameter required at the upstream concrete-lined sections, namely, 16 feet, there was adequate space on each side of the steel liner for the above operations (Figure 11). In addition, adequate space was available to thoroughly consolidate the concrete which was placed in one operation between the steel liner and the rock over the entire 328 feet length of each penstock steel liner. To facilitate the work of grouting outside the penstock steel liners, tapped holes 2 inches in diameter were formed in the steel liner during fabrication and steel plugs were supplied to close these holes when grouting had been completed. The holes were drilled in rings at 8-foot centres along the length of the liner, there being four holes to each ring equally spaced on the circumference. All grouting operations were carried out through these holes.

The grout program as planned consisted of

1. Low pressure grouting at 100 psi through holes drilled through the concrete and two feet into rock, the grouting nipple being fitted into the hole in the steel liner. This grouting operation was designed to ensure proper contact between the concrete and rock at the tunnel soffit, and also to fill any gap which might exist between the steel liner and the concrete.

2. High pressure grouting at 300 psi through holes drilled up to 15 feet into rock. This operation was designed to consolidate any areas of rock which may have been shattered during excavation of the penstock.

The program as set out above was carried out in its entirety for penstocks No. 1 and No. 2. However, on the basis of this experience it was decided to omit the second, high pressure grouting stage since very little grout could be injected into the surrounding rock. For penstocks No. 3 to No. 8, therefore, the grouting was carried out in one stage at 100 psi.

The construction operations described above were carried out smoothly

and efficiently, and in no way did they interfere with or delay completion of the power house. However, because of the urgent demand for power, it was impossible to carry out any tests for watertightness except by filling the conduits and making observations, especially at the upstream wall of the power house, for possible leakage.

The filling operation was carried out in stages, observations being made at the end of each stage to check for leakage. No. 1 penstock was filled first, and penstocks Nos. 2 to 8 were then inspected. Thereafter all penstocks were filled, and the supply tunnel was filled in stages over one-third and then over two-thirds of its length. Finally the entire tunnel was filled, and the pressure conduits subjected to full static pressure.

During the last stage of the tunnel filling operation, a severe leak was observed at an access door which had been installed through the concrete plug of the downstream adit, and it was necessary to drain the tunnel and stop the leak. When this work had been completed, the entire system of pressure conduits was filled in one operation without incident.

The Bersimis No. 2 development has now been in operation since October, 1956, and the only indication of possible leakage which has been observed is dampness and very small leaks at the upstream wall of the power house.

Bersimis No. 2 Pressure Conduits

The design of pressure conduits at Bersimis No. 2 followed construction at Bersimis No. 1, and therefore was influenced by experience gained at the latter site. The arrangement of pressure conduits adopted at Bersimis No. 2 may be seen on Figure 13.

The tunnel will be concrete lined and circular, and will have a finished diameter of 38 feet. At the manifold the tunnel will branch into five 17-foot diameter conduits which will reduce to 12 feet in diameter just upstream from the rotary valves in the power house. The economic studies to determine the conduit diameter near the power house were influenced considerably by the cost of these large valves.

The rock along the route of the pressure conduits may be identified as paragneiss, with intrusions of massive granite at the downstream end. In general, the gneisses provide good tunnelling conditions, although the rock is somewhat blocky. The minimum rock cover over the pressure conduits was set at 50 per cent of the maximum internal pressure head, using the same considerations as applied at the Bersimis No. 1 development.

The tunnel profile was influenced to a large extent by the presence of a valley upstream from the surge tank. The requirement of 50 per cent cover at this valley established the almost horizontal grade of the 17-foot diameter branches. Tunnelling is being carried out almost entirely from the downstream end, and the 30-degree slope to the lower level in the tunnel was therefore located just downstream from the intake to facilitate construction.

Steel liners are to be installed in the conduits downstream from the manifold where the rock cover is less than 50 per cent of the maximum internal pressure head which amounts to 510 feet.

The closing of the rotary valves will exert tension in the steel liners, and for a distance of 48 feet upstream from the power house the steel liners will be maintained at a constant diameter and treated on the outside surface to prevent the transfer of stress to the surrounding rock. Transition to a

diameter of 17 feet occurs immediately upstream from the coated section and extends to a point about 100 feet upstream from the power house. Up to this point, the steel liner is designed to withstand the full internal pressure within the usual design limits for steel. This requirement was determined on the basis of a stability study of the rock slope, taking into account the hydrostatic forces due to ground water and the thrust on the penstocks when the rotary valves are closed. Continuing upstream, the steel plate thickness was reduced until, on the basis of the design, the rock supports 50 per cent of the internal pressure. The stress which would occur in the steel at this point, without rock support, amounts to 30,000 psi, and this limitation on stress applies for a distance of 100 feet upstream. The rock cover over this 100-foot length varies between 30 and 35 per cent of the internal pressure head, and thereafter the design of the steel liners was governed by the external pressures which could develop during a dewatering operation. The portions of the steel liners which were designed for the condition of external pressure amount to 40 per cent of the total length of the liners.

It was considered that the maximum external pressure head during dewatering could, under some circumstances, rise to an amount equal to the depth of rock cover over the steel liner and the design was on this basis, no drainage works being provided. The necessary thickness of the steel liner to withstand this external pressure was calculated using the curve given on Figure 4, for which it is assumed that a gap equal to three ten-thousandths of the conduit radius will exist between the steel and the surrounding concrete. It is proposed to grout outside the liner and between the concrete lining and the rock to a pressure of 50 psi to ensure that the gap will not exceed the design value. It should be noted that the grouting pressure of 50 psi is a substantial proportion of the critical external pressure for the steel liner.

The same considerations as applied at the Bersimis No. 1 development, as regards the type of steel to be used in the liners, also applied at Bersimis No. 2. The liners will be fabricated from steel meeting ASTM Specification A201, grade B, firebox quality, and will be shipped from the manufacturer's works in lengths of approximately 50 feet. These sections will be unloaded at the power house and inserted in the penstock tunnels, using the power house crane.

It is expected that the methods of installation, inspection, concreting and grouting of the steel liners of the Bersimis No. 2 development will follow along similar lines to those employed at Bersimis No. 1.

Chute-des-Passes Pressure Conduits

The Chute-des-Passes development has been designed very recently and is now under construction. The supply tunnel will have an equivalent diameter of 34 feet 4 inches, and will be driven roughly parallel to the Peribonka River. It will have a total length from the intake to the manifold of 30,854 feet and will be concrete-lined throughout. The supply tunnel will branch at the manifold into five steel-lined penstocks having a diameter of 15 feet over most of their length. As the penstocks approach the power house, the diameter will be reduced in three stages to the diameter of the penstock valves, namely, 11 feet. The profile of the pressure conduits is given on Figure 14.

Rock types encountered along the route of the pressure conduits are again the paragneisses and massive granites of the Canadian shield and little

trouble is anticipated in tunnel driving. The tunnel line crosses a number of tributaries of the Peribonka River, and a minimum cover equal to 50 per cent of the maximum internal pressure has been maintained at these points. It is interesting to note that, if it had been decided to maintain a lower percentage cover, the length of the supply tunnel would not have been materially reduced.

The five steel-lined penstocks will be near horizontal, will have an average length of 325 feet, and will be subjected to a maximum internal pressure head of 920 feet. The design of the steel liners is similar to that used at Bersimis No. 2, and the external pressure which can occur when the conduit is dewatered controls the thickness of the liners over most of their length. The downstream 32-foot length of each penstock will be coated with emulsified asphalt to prevent adhesion to the surrounding concrete, and hence these sections of the liners are designed to resist the maximum internal pressure at a stress of 15,000 psi. For a distance of 64 feet upstream from these sections, the thickness of the steel liners is reduced to that which is required to resist external pressure during dewatering. The method of design against external pressure is the same as was used in the Bersimis No. 2 development.

The steel liners will be fabricated from steel to ASTM Specification A201, grade B, firebox quality. However, in this case, since the required steel thicknesses are greater than those used in any of the two developments previously described, it has been specified that the steel shall be produced using fine grain practice.

Charpy tests will be made on test specimens taken from the plates as well as from the shop welds as a protection against brittle failure. More extensive field welding will be necessary at Chute-des-Passes as compared with both of the Bersimis developments described previously, since it will be impossible to transport the 15-foot diameter sections of the liner in lengths greater than approximately 10 feet.

It is anticipated that the installation, inspection, concreting and grouting of the steel liners will be carried out in a manner similar to that used in the two Bersimis developments except that, in this case, it will be necessary to transport the steel liner sections into the penstock from the manifold, access to which will be obtained by a short adit driven from the main access tunnel to the power house.

ACKNOWLEDGMENTS

The data concerned with the Bersimis developments and the Chute-des-Passes development were made available, respectively, by the Quebec Hydro-Electric Commission and the Aluminum Company of Canada, Limited.

The authors wish to express their thanks to the engineers of the above-mentioned organizations who made possible the preparation of this paper.

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APPENDIX I

Stresses in a Steel Liner Due to Internal Pressure

Nomenclature Also see Figure 15.

E = Young's Modulus for steel

E_c = Young's Modulus for concrete

E_r = Young's Modulus for rock

b = radius to inside of steel liner

c = radius to outside of surrounding concrete

d = radius to end of radial fissures in rock

R = radius to point considered

e = thickness of steel liner

t = gap between steel liner and surrounding material

$1/m$ = Poisson's ratio for rock

p = pressure in conduit

p_b = pressure transferred by steel liner to surrounding concrete

p_c = pressure transferred from concrete to radially fissured rock

p_d = pressure transferred from radially fissured rock to solid rock

λ = proportion of pressure taken outside steel liner = p_b/p

σ_s = hoop stress in steel liner

P_z = vertical pressure of overlying rock

σ_t = tangential stress in rock above soffit of circular cavity

N = ratio of horizontal to vertical stresses in the rock before tunnel excavation

Determination of Hoop Stresses in Steel Liner Assuming Radial Fissures in Surrounding Concrete and Immediately Adjacent Rock

Consider firstly the deformation of the steel liner:

Pressure taken up by steel liner = $p - p_b$

Hoop stress in steel liner = $\sigma_s = (p - p_b)\frac{b}{e}$

$$= p\,(1 - \lambda)\frac{b}{e} \qquad (1)$$

Radial deformation of steel liner = $U_s = \sigma_s b/E$

$$= p(1 - \lambda)\frac{b^2}{Ee}$$

Consider now the deformation of concrete and rock outside the steel liner:

a) In the concrete

Tangential stress = 0

Average radial stress $= \dfrac{p_b + p_c}{2} = \lambda\,\dfrac{p\,(b + c)}{2c}$

Radial deformation $= U_c$

$$\approx \frac{p_b + p_c}{2}\;\frac{(c - b)}{E_c}$$

$$\approx \lambda\, p\,\frac{(c^2 - b^2)}{2c\,E_c}$$

b) In rock with radial fissures

Tangential stress = 0

Average radial stress $= \dfrac{p_c + p_d}{2} = \dfrac{b}{2cd}\,\lambda\, p\,(d + c)$

Radial deformation $= U_r$

$$\approx \frac{p_c + p_d}{2}\;\frac{(d - c)}{E_r}$$

$$\approx \frac{b}{2cd}\cdot\lambda\, p\cdot\frac{(d^2 - c^2)}{E_r}$$

c) In the sound rock beyond fissured zone

The thick cylinder theory applied to an infinitely thick shell gives

Radial stress at internal radius d = p_d compression

Tangential stress at internal radius d = p_d tension

where $p_d = \lambda p \frac{b}{d}$ (2)

Radial deformation at radius d $= U'_r$

$$= p_d \cdot \frac{d\,(m+1)}{E_r m}$$

$$= \lambda p \cdot \frac{b\,(m+1)}{E_r m}$$

The deformation of the steel liner U_s must equal the sum of the deformation $U_c + U_r + U'_r$ outside it plus any gap, t, between it and the surrounding material, that is:

$$P(1-\lambda)\frac{b^2}{Ee} = \lambda p\,\frac{(c^2-b^2)}{2cE_c} + \lambda p\,\frac{b}{2cd}\,\frac{(d^2-c^2)}{Er} + \lambda p\,\frac{b(m+1)}{E_r m} + t$$

From this equation the value of can be deduced as

$$\lambda = \frac{\frac{b^2}{Ee} - \frac{t}{p}}{\frac{b^2}{Ee} + \frac{b}{E_r}\,\frac{(m+1)}{m} + \frac{c^2-b^2}{2cE_c} + \frac{b}{2cd}\,\frac{(d^2-c^2)}{E_r}} \qquad (3)$$

The hoop stress in the steel liner σ_s can now be calculated by substituting λ in equation 1

$$\sigma_s = p(1-\lambda)\frac{b}{e}$$

Determination of Length of Radial Fissures in Rock Surrounding a Steel-Lined Pressure Conduit

Assume that the radial fissures extend to the point where the natural compressive stresses in the rock are just exceeded by the tensile stresses caused by internal pressures in the conduit.

Terzaghi and Richart(3) give the compressive tangential stresses in the rock above the soffit of a circular cavity as

$$\sigma_t = \frac{Pz}{2}\left(1+\frac{c^2}{R^2}\right) - \frac{Pz}{2}\left(1+\frac{3c^4}{R^4}\right) + N\left[\frac{Pz}{2}\left(\frac{1+c^2}{R^2}\right) + \frac{Pz}{2}\left(1+\frac{3c^4}{R^4}\right)\right]$$

where P_z = vertical pressure of overlying rock

N = ratio of horizontal to vertical stresses in the rock before tunnel excavation

R = radius to any point in the rock

The tensile tangential stress in the rock at the end of the radial fissure caused by internal pressure in the conduit is given by equation (2) $P_d = \lambda p \frac{b}{d}$

where
$$\lambda = \frac{\frac{b^2}{Ee} - \frac{t}{p}}{\frac{b^2}{Ee} + \frac{b}{E_r}\left(\frac{m+1}{m}\right) + \frac{c^2-b^2}{2cE_c} + \frac{b}{2cd}\,\frac{d^2-c^2}{E_r}}$$

as given in equation (3)

By the opening assumption the radial crack will extend to radius d equal to R where p_d equals σ_t.

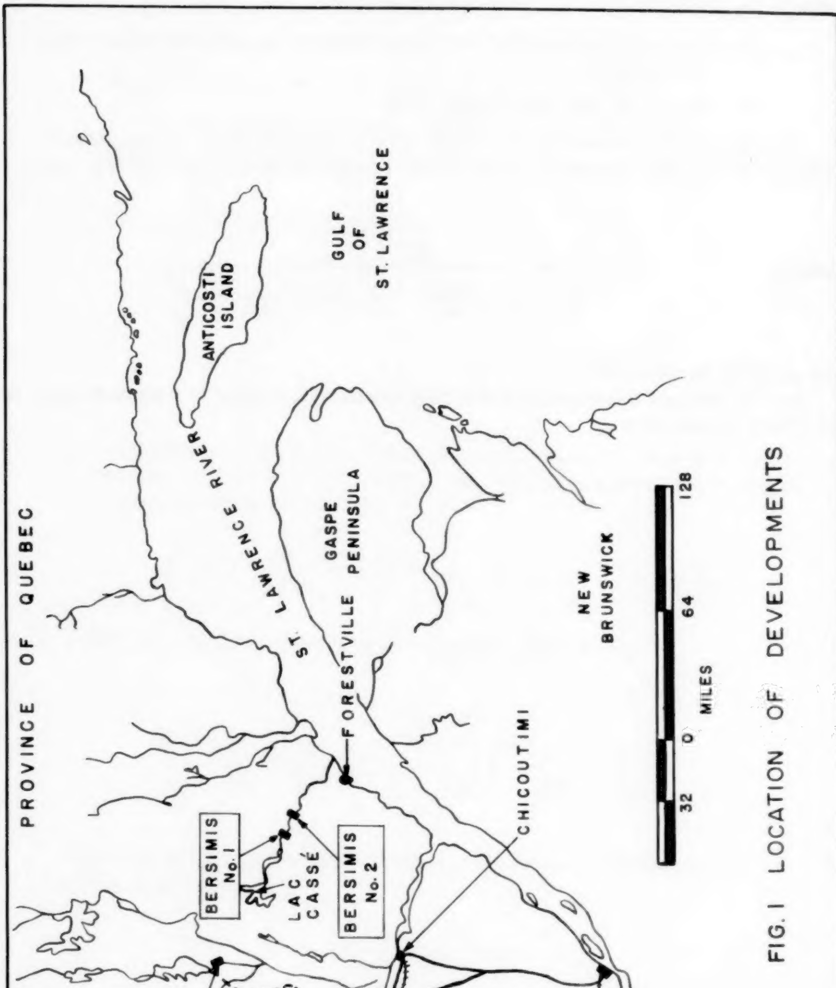


FIG. I LOCATION OF DEVELOPMENTS

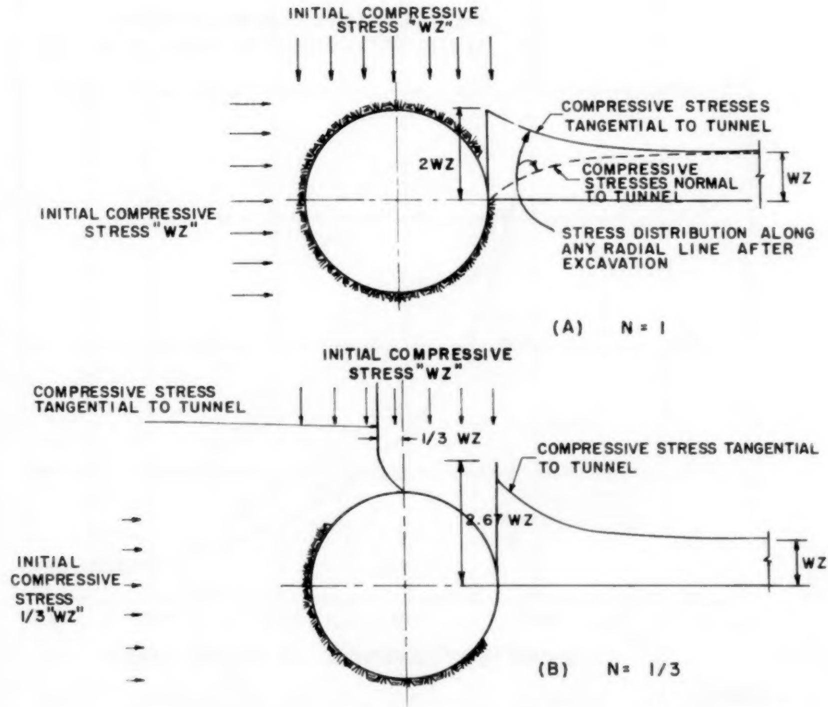


FIG. 2 ROCK STRESSES AFTER EXCAVATION

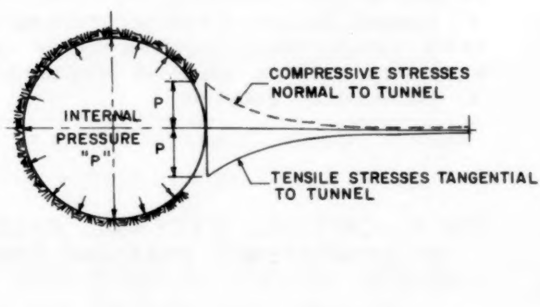
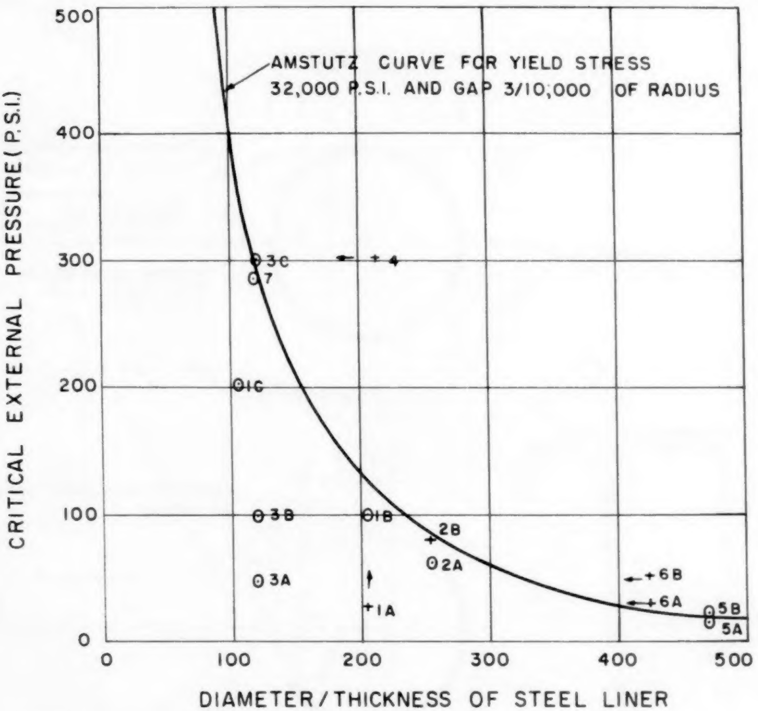


FIG. 3 ROCK STRESSES INDUCED BY INTERNAL PRESSURE



LEGEND
+ BUCKLING OF LINER DURING GROUTING OR DEWATERING
⊙ CONDUIT GROUTED OR DEWATERED WITHOUT BUCKLING

KEY TO PLOTTED POINTS
1A - SHIRA-ACCIDENTAL OVERPRESSURE (> 25 P.S.I.) DURING GROUTING
1B - SHIRA-MEASURED GROUNDWATER PRESSURE ON DEWATERING
1C - SHIRA-MAXIMUM SPECIFIED GROUTING PRESSURE
2A- WHATSAN-ESTIMATED GROUNDWATER PRESSURE ON DEWATERING
2B- WHATSAN-OBSERVED PRESSURE DURING FIRST STAGE GROUTING
3A, B & C- BERSIMIS, LAC CASSE - GROUTING STAGES 1, 2 & 3
4- KEMANO - SECOND STAGE GROUTING (METAL THICKNESS ≮ 9/16")
5A & B - LADORE FALLS - GROUTING STAGES 1 & 2
6A & B- NILO PECANHA - GROUTING STAGES 1 & 2 (METAL THICKNESS ≮ 9/16")
7- CALANCASCA - GROUTING

FIG. 4 - CRITICAL EXTERNAL PRESSURE FOR BUCKLING OF UNSTIFFENED PRESSURE CONDUIT STEEL LINERS

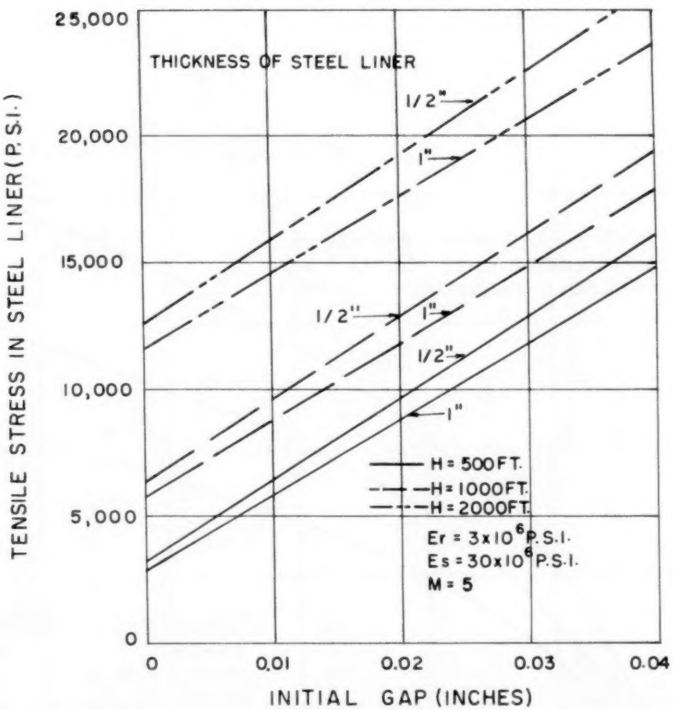


FIG. 5 EFFECT OF INITIAL GAP ON STRESS IN STEEL LINER DUE TO INTERNAL PRESSURE

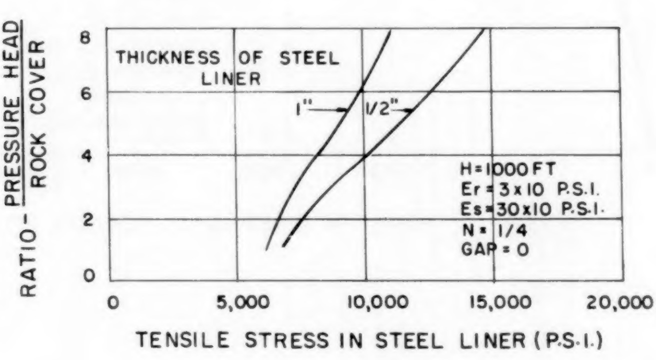


FIG. 6 EFFECT OF ROCK COVER ON STRESS IN STEEL LINER DUE TO INTERNAL PRESSURE

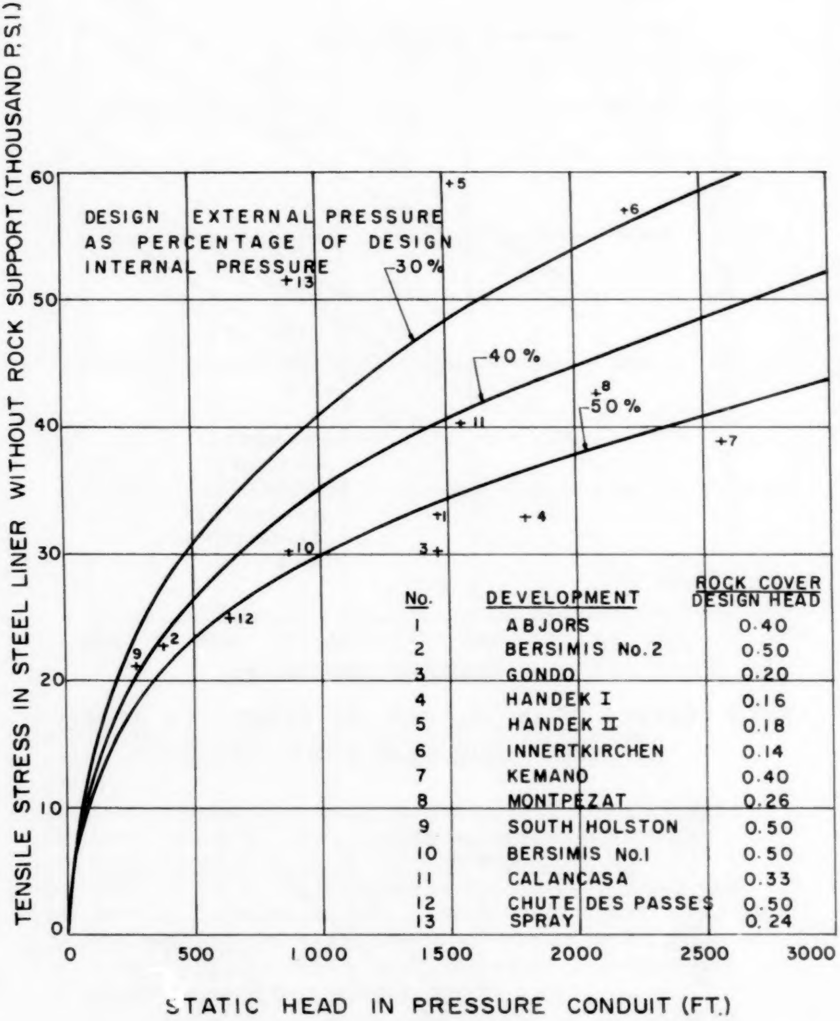


FIG. 7 INTERRELATION OF INTERNAL AND EXTERNAL PRESSURES ON STEEL LINERS

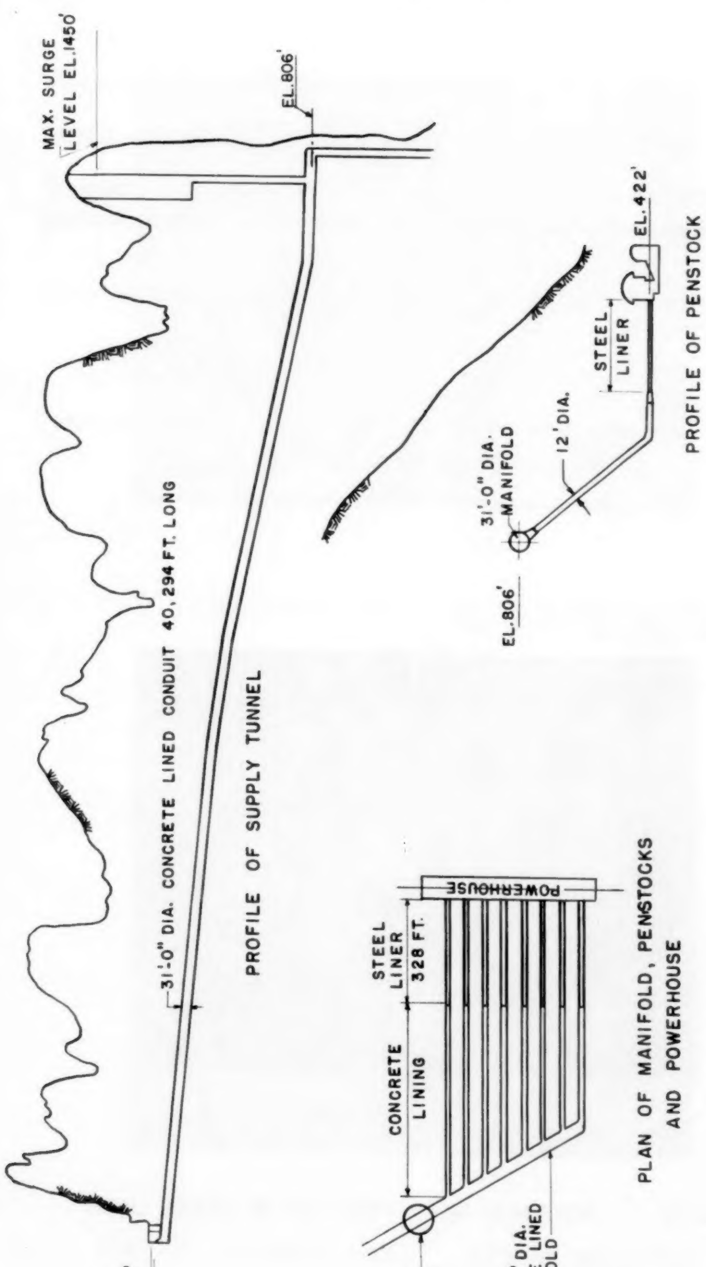


FIG. 8 PLAN AND PROFILES OF BERSIMIS No. I DEVELOPMENT

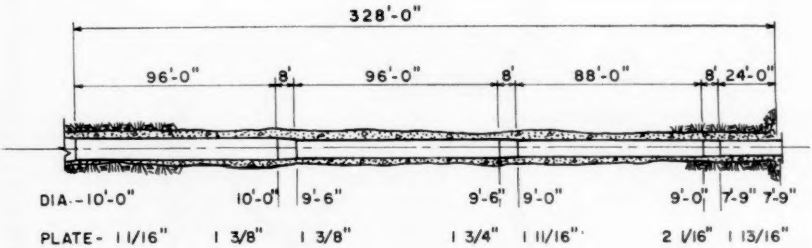


FIG. 9 BERSIMIS No. 1 PROFILE OF STEEL LINED SECTION OF PENSTOCK

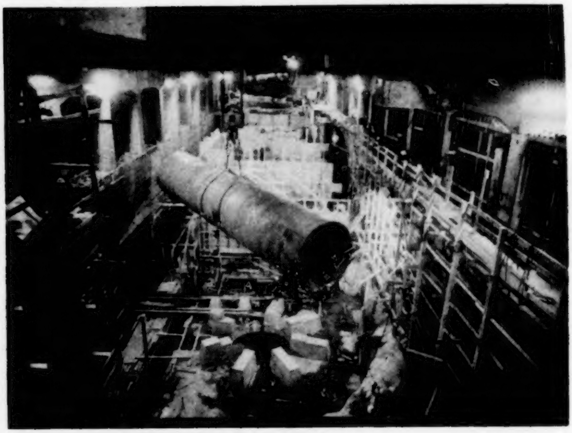


FIG. 10 BERSIMIS No. 1 SECTION OF STEEL LINER

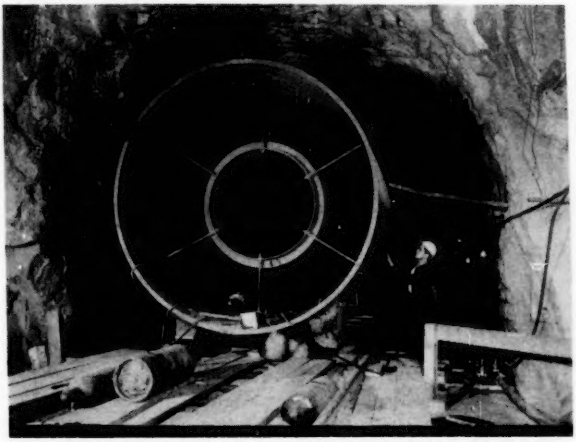


FIG. 11 BERSIMIS No. 1 STEEL LINER IN TUNNEL PRIOR TO CONCRETING

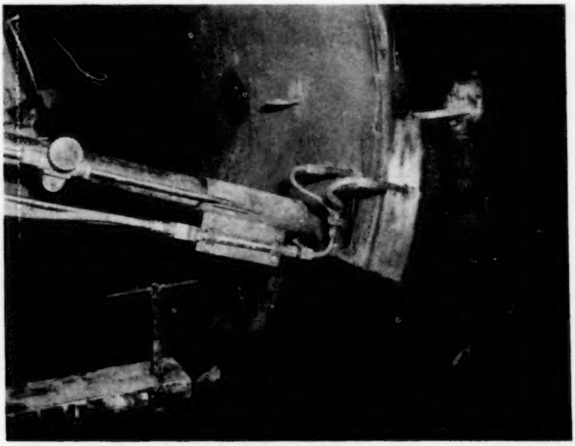


FIG. 12 BERSIMIS No. 1 STRESS RELIEVING CIRCUMFERENTIAL JOINTS IN STEEL LINER

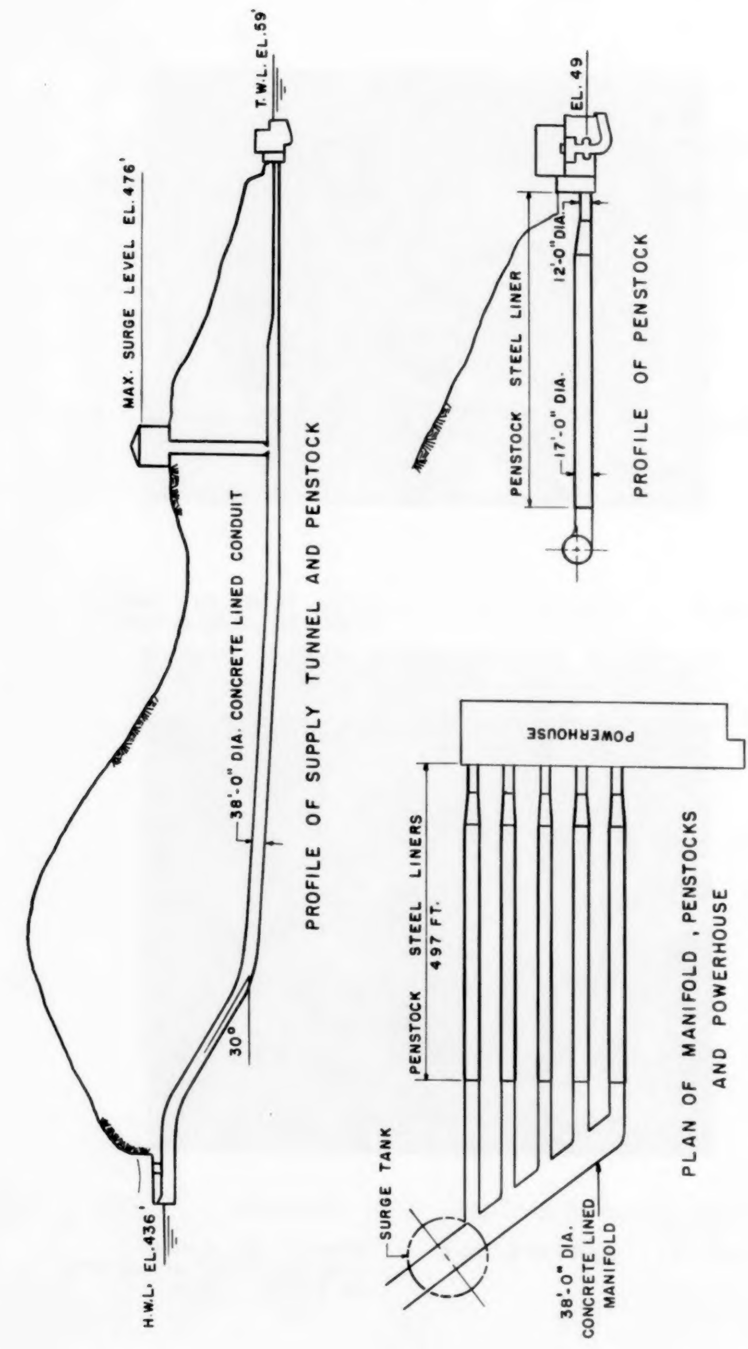


FIG. 13 PLAN AND PROFILES OF BERSIMIS No. 2 DEVELOPMENT

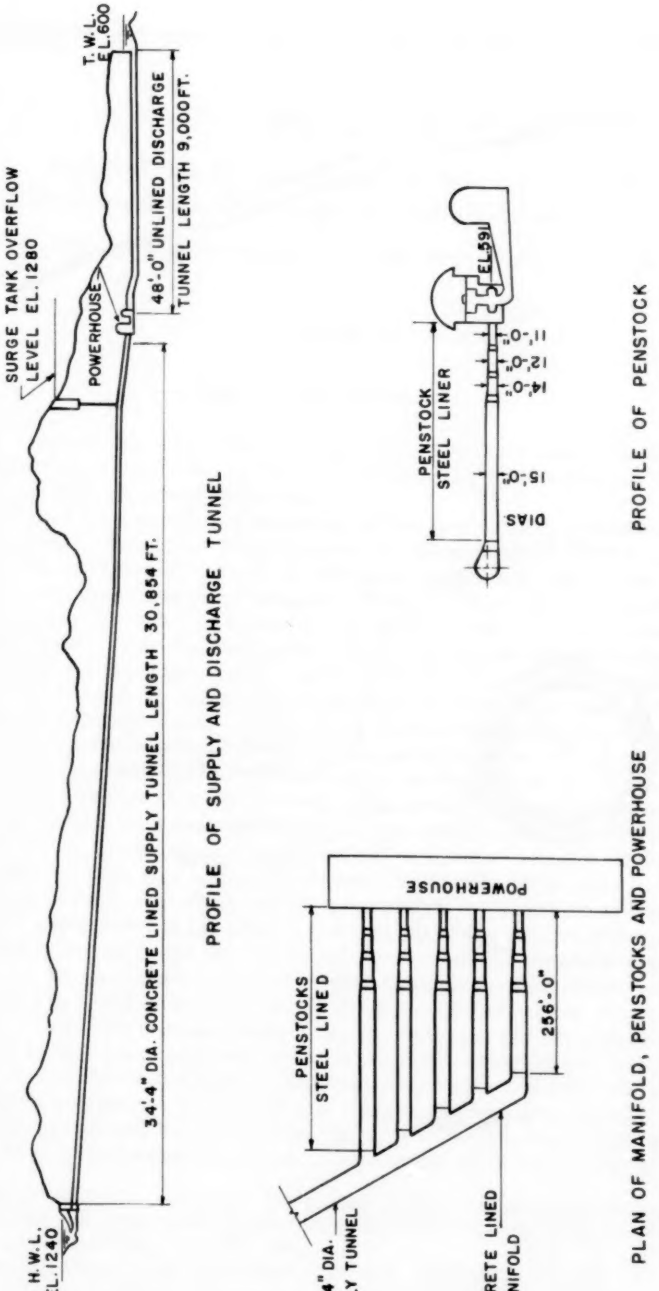


FIG. 14 — PLAN AND PROFILES OF CHUTE-DES-PASSES DEVELOPMENT

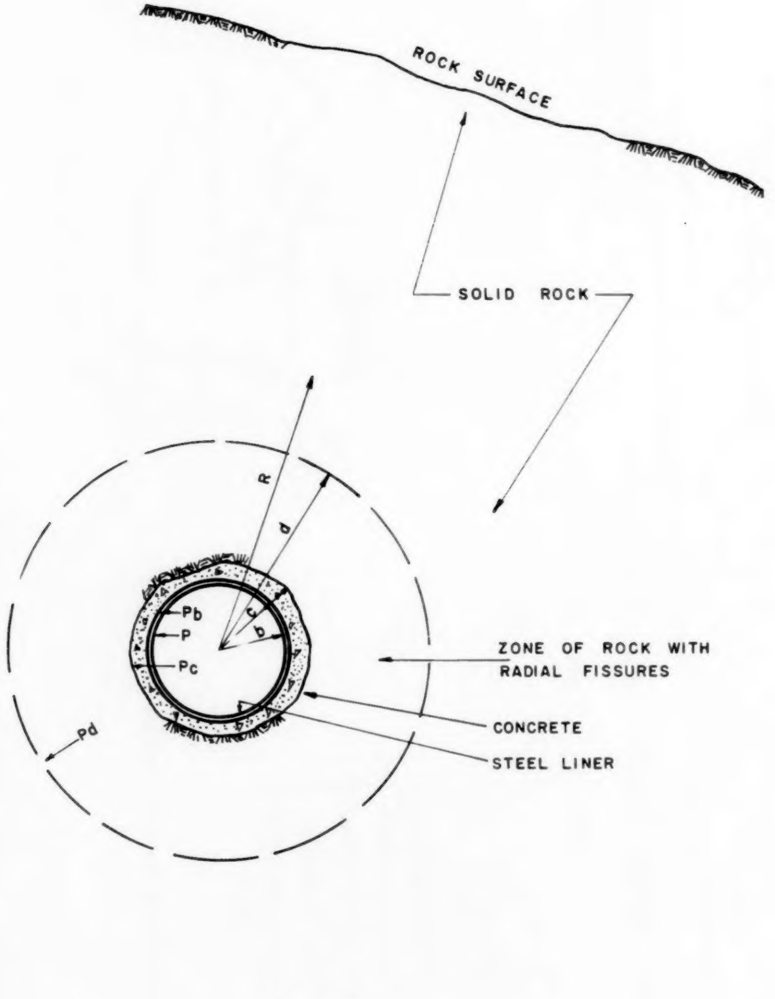

ROCK SURFACE
SOLID ROCK
R
d
c
b
Pb
P
Pc
Pd
ZONE OF ROCK WITH RADIAL FISSURES
CONCRETE
STEEL LINER

FIG. 15

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Proceedings of the American Society of Civil Engineers

FACTORS IN SELECTION OF PIEVE DI CADORE AND FEDAIA DAMS[1]

Carlo Semenza,[2] M. ASCE
(Proc. Paper 1458)

Foreword

We give below a brief outline of the criteria we adopted in the choice of the Pieve di Cadore Dam on the River Piave, and Fedaia Dam on the Upper Avisio (both in the Eastern Dolomites).

Each dam problem constitutes an individual case: therefore no generalization—tempting as it may appear—is advisable in a study of this kind. First of all, in the case of Pieve di Cadore a preliminary choice was made of all possible locations to be dammed. This was followed by a rapid but exhaustive examination as to the suitability of the locations, from the morphological, geognostic, static and economic point of view, and the functional use of the reservoir and power plant. In the case of Fedaia this examination proved not to be necessary. As a first result, the best site was chosen and in both cases more extensive investigations were carried through on it. Finally, a diligent search was made for the type of dam structure best suited to the location. The latter investigation was carried out according to our usual system by developing alternative plans with enough details to permit a reliable cost estimate. The details included also a check of the actual operation and maintenance costs foreseen for the project, since experience has demonstrated that the true efficiency of a proposed design reveals itself only after many years of operation. This method of study is long and expensive, but again experience has amply demonstrated that the subsequent savings in the constructional and operational stages far outweigh the increased expenses at the project stage.

Both dams were designed by the Construction Department of Societa Adriatica di Elettricita (S.A.D.E.) in collaboration with Prof. F. Arredi, A. Danusso and G. Oberti and built under the supervision of the Construction Department. As for the dates of construction, it must be remarked that one of the two dams —that at Pieve di Cadore—has been in operation since the end of 1949. Studies were commenced in 1939-40 and completed soon after the war. Therefore this report will refer to the technique in force ten years ago, which has meanwhile made further progress. On the other hand, the Fedaia dam is quite

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1. Abridged from "L'Acqua", nos. 5-6, 1955 and 7-8, 1956.
2. Director, Hydr. Dept., S.A.D.E., Venice, Italy.

recent. It was put into operation in 1956, and the pertaining research work has been carried out in the last five years.

The Pieve di Cadore Dam

Choice of the Location

The exploitation of the River Piave had been prospected together with that of its tributary Boite. The best solution from the point of view of storage capacity, could be obtained by placing the dam as close as possible to the confluence of the rivers. Downstream from the confluence the conditions were not favorable. Therefore we carefully examined the geological and morphological conditions of the river upstream from the confluence, where ten possible dam locations were selected. These were subjected to systematic investigations—geological surveys, borings, wells, etc.—which went on for a few years.

Finally the choice fell on the location called Pian delle Ere which proved to be the least unfavorable. The efficiency of a solution in the range of a given plant is not only dependent on the characteristics of the dam as a separate structure, but also on many other elements, from which the general economy of the exploitation does actually result. In the present case the net storage capacity and lesser extension of the other plant works in relation to the site of the dam—especially length and diameter of the conduits—were more favorable to the choice of Pian delle Ere than of the other admissible locations.

Characteristics of the Selected Location

From the morphological point of view, the problem of damming the River Piave at Pian delle Ere was characterized by two difficulties: 1) the site selected was markedly asymmetrical (Fig. 1), owing to the presence, on the right side, of a deep, narrow gorge, while the left side was constituted by a large horizontal plateau—the actual Pian delle Ere—ending downstream in a steep rocky wall; 2) the axis of the deep gorge was convergent to the general axis of the valley (Fig. 3, 4, 5) and moreover, the deep gorge increased in width as it neared the axis of the valley.

From the geological point of view, the doubts depended on the actual nature of the ground below the horizontal plateau which presented numerous rocky outcrops and a slight depression filled with gravel on the left side, which seemed to correspond to an original river bed. Systematic borings and wells on the plateau showed that it was constituted mainly by compact rock, which in the bottom of the left side depression sank only a few metres below the surface. At the same time, the gravel in the bottom of the gorge was found to be rather thin by means of several borings that demonstrated also the complete absence of deep narrow cuttings which are very frequent in valleys from limestone erosion. The surveys went on for two working seasons (1937-1938). After ascertaining the geological constitution of the location, we turned to the technical and economic problem of the structure to adopt for the dam.

Choice of the Type of Dam

The morphological conditions suggested one of the following solutions:

(a) A composite dam, consisting of an arch at the right viz. in the deep

gorge; the arch would abut to the right on the rock, to the left partly on the rock and partly on an artificial support or massive buttress. This would be followed by a gravity wall on the plateau (Fig. 3);

(b) A rectilinear gravity dam, either massive or lightened, between the two rocky spurs delimiting the location of least width;

(c) A curved dam, massive or lightened. (Fig. 4)

Solution (a) was the first to be examined. Research by model demonstrated that, on account of the large dimensions required for the buttress, the total cost of the dam would exceed the cost of the other solutions. In fact, the breaking and shearing stresses at the base of the buttress would be so high that in order to withstand them the base of the buttress, and consequently the volume, would be considerably increased, thus nearing the edge of the plateau downstream and endangering the static conditions of the whole structure.

The same difficulty was met in the study of the second solution; owing to the marked convergence in plan of the deep gorge to the general axis of the valley, the downstream toe of the dam portion that was to rest on the Pian delle Ere plateau would have had to stand at a little distance from the edge of the same, the downstream toe of some blocks having to be deeper by about 50 m than the upstream one, so that the foundation of that portion of the dam would have extended to the very bottom of the gorge, with a considerable increase in excavation and masonry. The adoption of a lightened structure, always on a rectilinear plan, would have made the situation worse, for it would have called for a greater base thickness.

The third solution—i.e., a curved structure—originated from the above considerations. Actually, a gravity dam designed between the two rocky spurs we have mentioned, was bound to assume a curved plan in order to satisfy the three following requirements:

(1) Cross the deep gorge with a "plug", the axis of which was to be nearly at right angles to the gorge, so that the plug could be in favorable static condition;

(2) Cross the deep gorge following an axis as far upstream as possible, since its width increased downstream;

(3) Push as far upstream as possible the downstream toe of the dam portion resting on the plateau, as compared to the downstream plateau edge.

Thus arose the conception of an arched dam. The morphological conditions of the location suggested a curvature of considerable value for static purposes, and so we came to the study of an arch-gravity dam. Its characteristics were extrapolated in regard to existing arch-gravity structures. In fact, overlooking the deep gorge on the right side which would be blocked by an independent plug, the ratio of chord (308 m) to height (about 55 m) was over 5.6, as against 4.2 of the Gibson dam, which represented the most advanced example of this kind of structure.

Extensive analytical studies and tests on a large-scale (1:40) model were carried out.

At the same time, studies on gravity structures, also curved but not utilizing the curvature as an element of their stability, went on. In particular, a curved structure was considered, constituted of hollow blocks, statically independent. An extensive comparison was made between the arch-gravity solution and the hollow-block solution.

The results of the comparison, extended to several designs on different

plans, were markedly favorable to the arch-gravity type, even from the economic point of view alone, i.e. overlooking the static superiority of the arch-type in exceptional stress conditions. In fact:

(1) Owing to the widening of the deep gorge downstream, the greater base thickness of the hollow type in comparison with the arch-gravity one, entailed "ipso facto" considerable increase of volume for the plug;

(2) It would not have been advisable to build side by side blocks of different heights (up to a 1:2 ratio): whence the necessity of blocking the deep gorge by a massive structure, at least from the bottom up to a certain elevation.

To conclude, the arch-gravity solution (Fig. 1, 2 and 5), consisting of a massive arch resting on the right on a "plug"that blocked the deep gorge, and on the left on the rocky plateau, proved to be the most suitable and therefore the most economical. In figures: a volume of not less than 480,000 m^3 would be required for the gravity solution, while the volume of the arch-gravity one did not exceed 377,000 m^3. The saving of about 100,000 m^3 of concrete led to a reduction of at least 15% in the total cost.

It may be added that 5 years of operation have confirmed the perfect efficiency of the chosen solution also from the operation point of view.

Fedaia Dam

Characteristics of Reservoir and Choice of Location

The Fedaia reservoir was formed by damming the Pian della Fedaia, a large plateau situated at the foot of the Marmolada massif, in the Dolomites, at about 2,000 m a.s.l. A concrete buttress dam was built on the western side of the plateau, and an earth dam on the eastern one. Here is a brief outline of our studies relating to the former.

To the west, the Fedaia plateau is bordered by a rocky sill deeply cut in by the torrent Avisio. The sill practically dictated the location of the dam, so that its choice involved no special problem.

We may remark incidentally that the main difficulty for the Fedaia reservoir was the complex geology and morphology of the plateau, which concerned the possibility of forming a water storage, and the volume of same. This study in various stages would be too long and beyond the scope of this article.

Character of Site

The main characteristics of the site (Fig. 6 and 7) from 1,995 m a.s.l. to 2,055 m—maximum elevation foreseen for the dam—were the following.

The left side of the location—towards the Marmolada massif—was formed by a rather steep rocky wall from 1,990 to 2,045 m; further up the side assumed an almost level trend for approximately 30 m. The right side went up almost vertically till approximately 2,010 m, after which it followed a long, gentle slope up to about 2,040 m; then it continued in an almost horizontal trend for about 300 m till it reached the foot of the mountain range that bounds the Fedaia plateau on the north.

The irregular profile on the right side constituted the main difficulty in a satisfactory solution.

The shape of the location suggested two different types of structure:

(a) An arch-gravity dam, supported by the rock up to about 2,040 m elevation, and by two 15 m high gravity walls from 2,040 m upwards;
(b) A gravity dam, either massive or hollow, or a buttress dam, whose foundations had to follow more or less the winding profile of the site in order to reduce the volume to a minimum.

Arch-Dam

In the case of an arch solution, the topographic conditions led to an arch having the following characteristics:

(a) On account of the remarkable difference between the slopes of the two sides, the dam ought to be assymetrical in order to avoid extensive excavations.
(b) Rather slightly curved, so that the central portion of the arch would not be founded too far eastwards; in fact on that side the rocky bottom of the Fedaia plateau sinks rapidly. Besides, as the contour lines were not very convergent going downstream, the connections at the haunches of the arch should have been studied with a rather ample radius and consequently small central angles, in order to obtain a good impact of the structure on the rock.

These considerations led us, at first, to design an arch-gravity dam with rather massive cross sections, which resulted in large volumes. Extensive model tests were carried out on this first design at the I.S.M.E.S. (Experimental Institute of Models and Structures) in Bergamo. Tests suggested the adoption of thinner types, in which a greater curvature was obtained chiefly by reducing the thickness of the crown cross-sections, i.e. increasing the ratio of abutment sections to crown sections.

A characteristic element of the arch dam was the lack of rocky supports in the upper 15 m of the arch. Model tests proved that this did not affect considerably the static conditions of the structure except, to a certain extent, its upper portion: so that two gravity walls could easily have been substituted for the rocky supports. This was a very important conclusion, which may prove useful in other cases.

Of some interest were also the failure tests in which load values were reached up to 5 times the normal. With this load value the model showed some cracking along the downstream perimetral joint. With higher loads, the right side slid slowly along the perimetral joint, soon followed by the left one. Finally, shortly before collapse,—with a load 11 times normal—the artificial supports were pushed in a direction tangential to the upper arches. These results, of course, must not be considered as absolute from the quantitative standpoint, as in failure tests many conditions alter their meaning; yet they indicate a remarkable and fully reliable resistance.

Buttress Dam

Contemporaneously with the arch solution two gravity structures were investigated: a conventional gravity dam and a buttress one. An analysis of the comparison between these two types is given in the appendix. The buttress dam soon proved to be more economical although some characteristics of the location were not favorable to it. In fact to carry out the most convenient types of buttress dams, the vertical component of the water load has to be taken into account, which requires upstream and downstream face inclinations

reaching values of 4-1/2 horizontal to 10 vertical and still more. In the present case, the acceptable depth of the rock foundation of a long portion of the site was rather limited, which caused us to discard these forms as being too wide at the base. Moreover, we were compelled to design a curved plan dam in order to follow the highest zones of the site with the foundation line of the dam, while maintaining—downstream of the dam—a countersloping rocky step, contributing to safety against sliding. Finally, to ensure longitudinal stability, the round-heads were given large dimensions while smooth, ample connections were provided between head and buttress in order to avoid stress concentrations. To comply with all these requirements, we were induced to study the type of buttress shown in Fig. 7 which seemed to meet all these requirements.

Besides the stability tests with the customary hypotheses, further tests were made on a 1:30 model of the buttress resting on a deformable foundation. The deformations shown by a first series of tests were so small that the tests had to be repeated under double loads in order to obtain more measurable values. The stresses on the faces were found to be in accordance with the results of the calculation.

The tests permitted various minor refinements.

Definitive Choice of Dam

The volumes of the two types described above were the following:

<u>Buttress Type</u>

Buttress Structure		143,500 m^3
Side Walls		16,500 m^3
	Total	160,000 m^3

<u>Arch Type</u>

Arch-Dam		157,000 m^3
Artificial Supports		3,500 m^3
Side Walls		16,500 m^3
	Total	177,000 m^3

The cost per m^3 can be considered the same.

Since the difference between the two volumes was only 10%, we decided to adopt the buttress dam, because it was quite satisfactory from the static point of view, and the more economical, although with fewer resources in case of exceptional loads.

We may add that the good elastic behavior of the curved structure and the regularity of its deformations suggested to us the idea of carrying our studies further. We believe that we could have succeeded in decreasing its volume without any substantial static alteration, thus obtaining approximately the same volume as that of the buttress dam, while building a structure with higher efficiency in case of exceptional loads. These provisions, however, would have led to the adoption of a dam, which, in spite of its high chord-to-height ratio (approximately 7.2) would have to operate mainly as a pure arch: the very unusual characteristics of this structure would have called for laborious and delicate analytical investigations and model-tests in order to clear any doubt that might have arisen with the authorities and ourselves.

A brief examination showed that the additional investigations would have required at least two years. Therefore, in consideration of the results already

obtained, we decided not to wait any longer and adopted the buttress solution.

Appendix

In the case of the Fedaia dam, a comparison between the costs of the buttress structure and the gravity one can be outlined as follows:

(a) The volume of concrete employed in the buttress structure was reduced by 24% in comparison with a conventional gravity dam, for which an amount of 210,000 m^3 of concrete was estimated necessary.
(b) Each cubic meter of concrete in the buttress dam required 0.31 m^2/m^3 of forms, against an amount of 0.27 m^2/m^3 required in our case by a gravity dam, with an increase of 15% Standard Blaw-Knox type of steel forms have been employed.
(c) The amount of cement in the mix, for which we employed a limestone aggregate, was 230 kg/m^3, in order to obtain a frost resistant concrete, as the dam was constructed on the Alps at 2,000 m a.s.l., where the temperature in winter easily reaches -20° C. From field tests of different mixes we reached the conclusion that an ordinary gravity dam in the same conditions could have been built with only 200 kg/m^3 of cement. The 30 kg/m^3 more that we employed in the actual structure represent an increase of 15% in the total cost of cement.

The net saving realized in that way can now be quickly obtained considering that the cost of such a structure can be roughly divided into three equal parts, i.e. labor, construction equipment, and building materials.

The final balance of cost of some recent dams constructed in Italy clearly showed that the erection and dismantling of forms represent a part of about 10 to 20% of the labor expenses; consequently, the use of a higher percentage of forms has—at worst—brought us to an increase in the labor cost that can be calculated as follows:

$$0.15 \times 0.33 \times 0.20 = 0.01 = 1\%$$

The increase in capital cost for the provision of a larger amount of steel forms was quite negligible by comparison with the cost of the other construction equipment needed; even with a quite unfavorable estimate and considering the forms cost as high as 20% of the total equipment cost, it is easy to realize that a 15% increase on that item leads to another increase of only:

$$0.15 \times 0.33 \times 0.20 = 0.01 = 1\%$$

in the final cost of concrete.

Considering now the influence of the increased amount of cement employed, that represents an increase in the cost of:

$$0.33 \times 0.15 = 0.05 = 5\%,$$

we reach the conclusion that our concrete costs us

$$0.01 + 0.01 + 0.05 = 0.07 = 7\%$$

to be added to the one for a conventional gravity dam built in the same conditions.

Should we now for instance assume that the price of the concrete for a usual gravity dam would amount to 25 $ /m^3, we could make the following comparison:

Cost of the usual gravity dam: 25 x 210,000	=	$5,250,000
Cost of the buttress dam 25 x 1.07 x 160,000	=	4,280,000
Net saving		$ 970,000

that represents an 18.5% saving of the cost of the old fashioned gravity dam.

fig. 1

PIEVE DI CADORE DAM
ON THE RIVER PIAVE
DEVELOPED ELEVATION
ALONG THE MIDDLE FIBRE OF THE ARCHES
FROM DOWNSTREAM

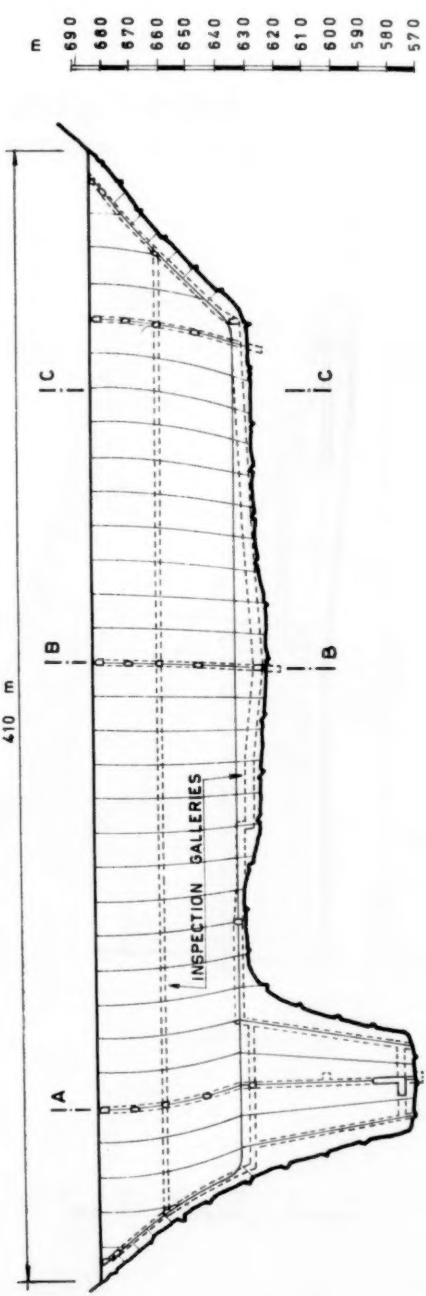
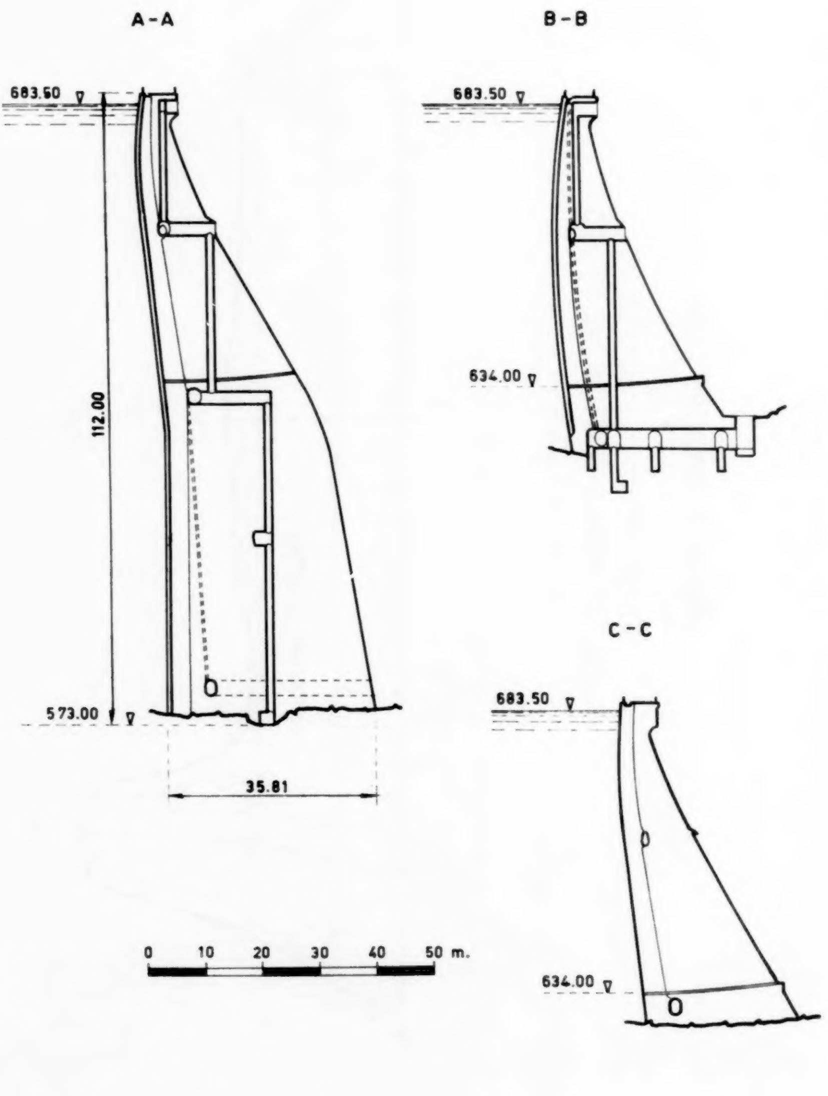


fig. 2

PIEVE DI CADORE DAM
CROSS SECTIONS



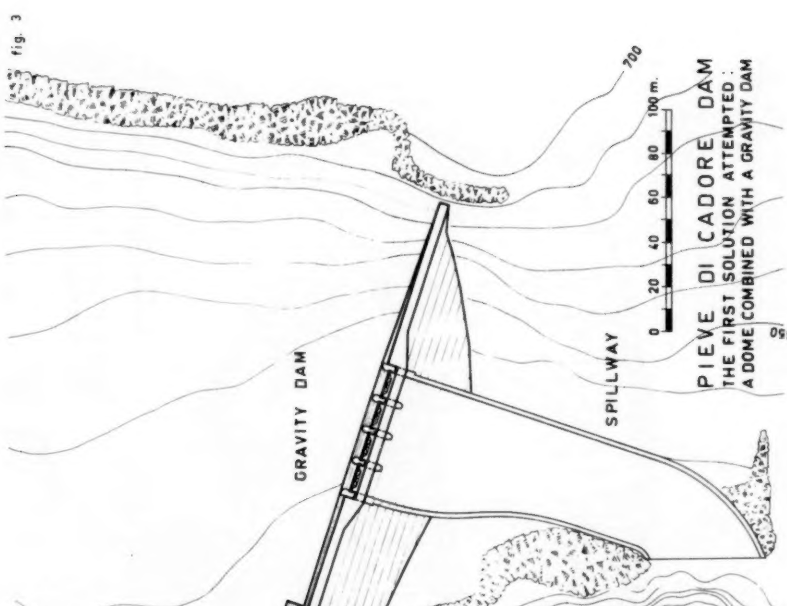


fig. 3

PIEVE DI CADORE DAM
THE FIRST SOLUTION ATTEMPTED:
A DOME COMBINED WITH A GRAVITY DAM

fig. 4

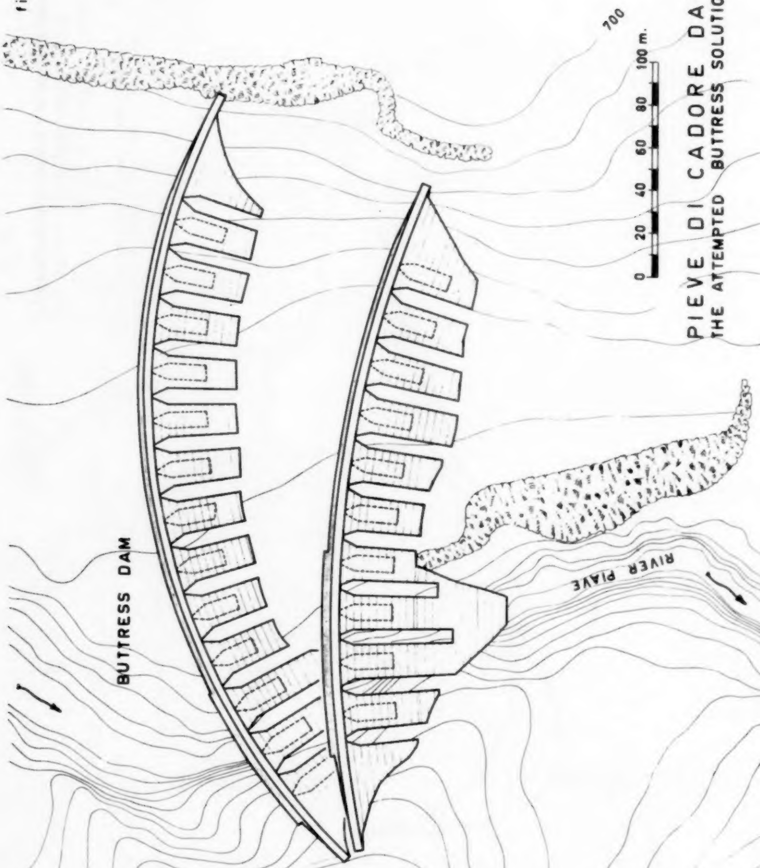

PIEVE DI CADORE DAM
THE ATTEMPTED BUTTRESS SOLUTION
0
20
40
60
80
100 m.
BUTTRESS DAM
RIVER PIAVE
700
650
600
600
650
700

fig. 5

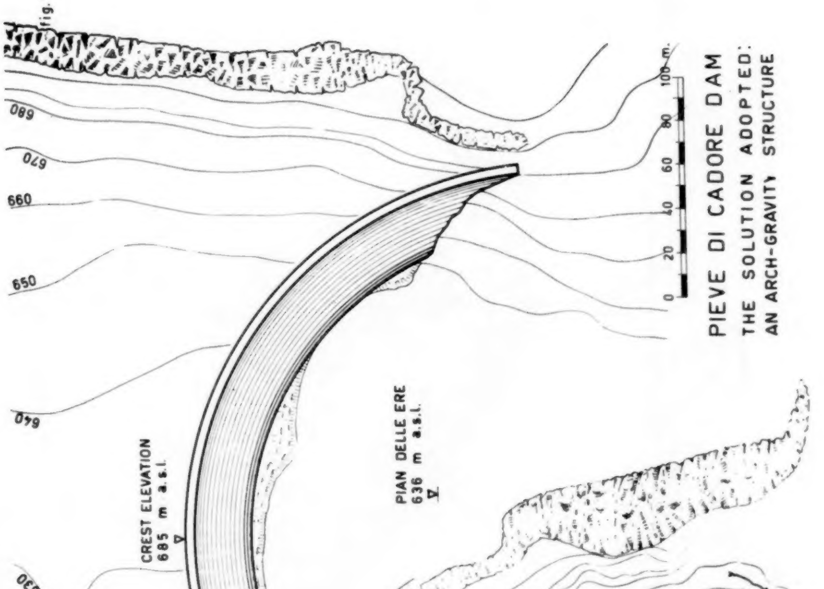
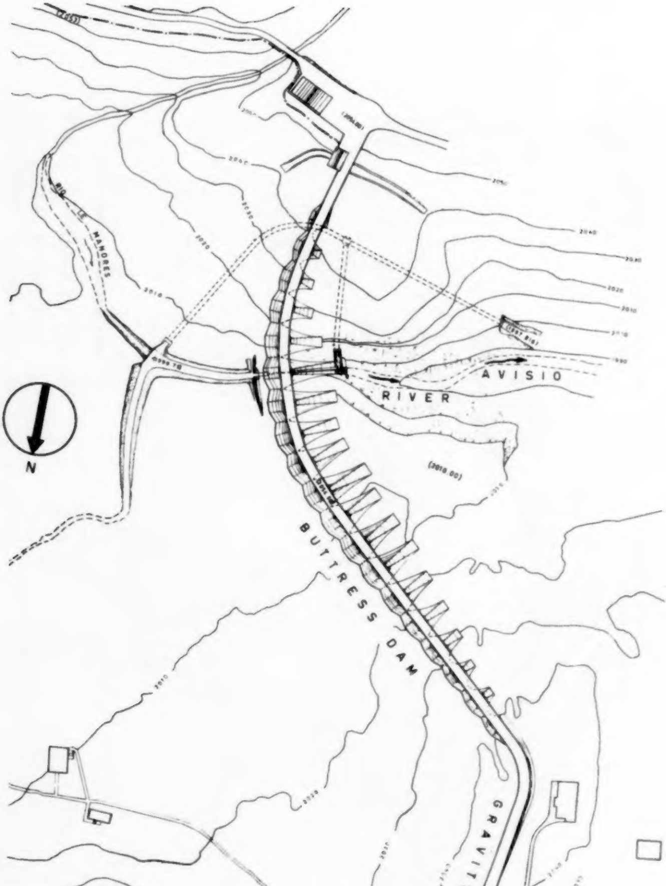
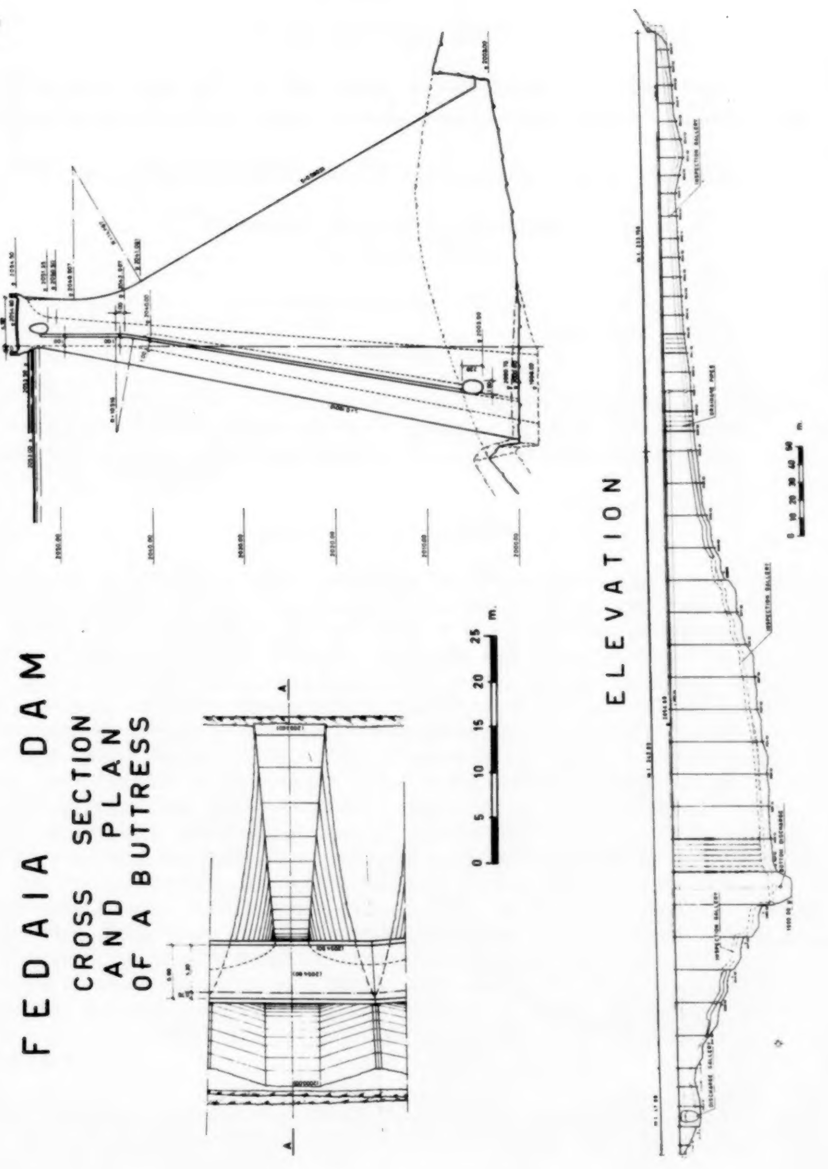

PIEVE DI CADORE DAM
THE SOLUTION ADOPTED:
AN ARCH-GRAVITY STRUCTURE
CREST ELEVATION
685 m a.s.l.
PIAN DELLE ERE
636 m a.s.l.
573 m a.s.l.
PLUG
RIVER PIAVE
0 20 40 60 80 100 m.
680
670
660
650
640
630
580
580
640
650

fig. 6



FEDAIA DAM
PLAN

fig. 7


FEDAIA DAM
CROSS SECTION
AND PLAN
OF A BUTTRESS
0 5 10 15 20 25 m.
ELEVATION
0 10 20 30 40 50 m.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

WATER RESOURCES AND WATER DEVELOPMENTS IN ALABAMA

Melvin R. Williams,* A.M. ASCE
(Proc. Paper 1459)

SYNOPSIS

The water resources of Alabama are evaluated and the increasing demands on the supply made by growth and development of the state are described. Problems of supply in dry years and for the future can be overcome by greater use of storage.

Interest in Alabama's water resources has grown with the passing of time and has paralleled the ever increasing growth and development of the State. In 1894 a start was made on the collection of stream-discharge information on the Tennessee River at Florence, Alabama, and a few years later a collection of similar information was begun on a number of other streams. These records were collected for only one purpose—to provide information for the development of water power. By 1914 the program was sharply curtailed on the assumption that enough information had been collected. A new program was started in 1928 to provide the Corps of Engineers with information needed in appraising the development potential of the larger streams in the State as part of a Nationwide program. Also at that time a start was made in cooperation with the State on a continuing investigation of the ground water.

Current discussions concerning Alabama's water resources involve developments of current interest such as water for supplying municipalities, industries, agriculture and recreational developments; water for the dilution and removal of wastes; water for hydroelectric power and navigation; the control of water during times of floods; and, finally, the development of an adequate and useful law governing the right to use water. All of these have one thing in common: they require that we have knowledge of our water resources.

Note: Discussion open until May 1, 1958, Paper 1459 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 5, December, 1957.

*Dist. Engr., U. S. Geological Survey, Montgomery, Ala.

The U. S. Geological Survey, in cooperation with several State and Federal agencies, collects information on the water resources of Alabama. This information shows that, although the State has water problems, Alabama is blessed with an unusually large supply of water. Annually, on the average, the State receives about 70 inches of rainfall at the coast and 50 inches in the interior, about 20 inches of which runs off in the streams.

How does the runoff in Alabama compare with the runoff in other States? Mississippi and the New England States have about the same runoff that Alabama has. Several smaller areas, especially in the Pacific Northwest and along the West Coast, have a greater amount, but most of the country has less. In the area between the 100th meridian and the Pacific Coast mountain ranges, the supply of water is small in comparison to that which Alabama has. East of the Mississippi River the supply is greater and in general is half or more than half of Alabama's supply. Thus, Statewide, Alabama takes second place to none in this resource.

The total runoff of the streams, which includes water that has moved through the ground and reappeared as streamflow, is a direct measure of the total supply available. Part of the water in transition through the ground is susceptible to capture through wells. In Alabama large amounts of ground water are available in limestone areas where underground solution channels exist, or in areas of unconsolidated geologic formations, such as those of the Coastal Plain. Elsewhere in the State, ground water is available in lesser amounts.

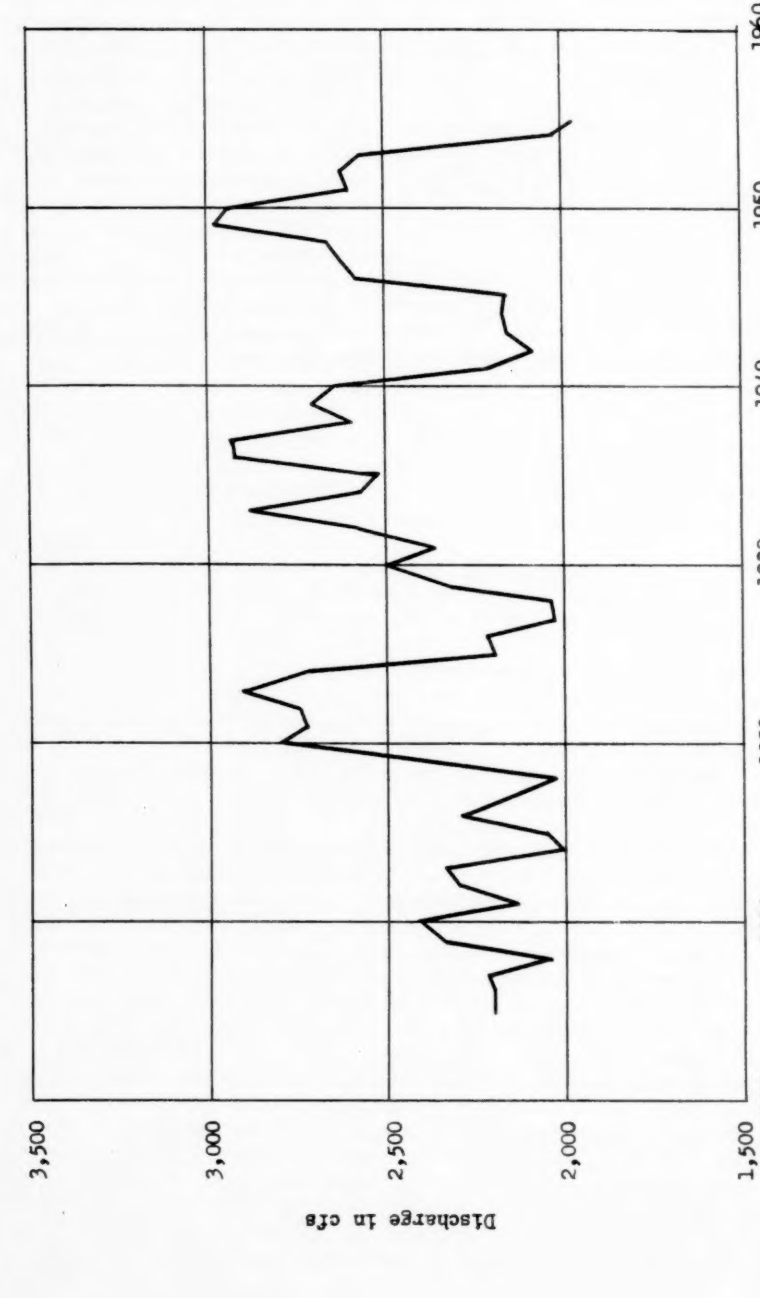
The supply of water in streams varies by years and by seasons. The figure of 5-year moving averages shows the trend in water supply of a representative Alabama stream for the period 1901 to 1955. The effect of the recent drier years is noticeable.

These dry years have brought with them problems of supply largely because they have coincided with a period of rapid growth in use of water. The combination of growth and short supply has forced many of Alabama's cities and towns to develop new sources of water at substantial cost. More water is being used for irrigating crops and pastures. New water-using industries have located in the State, bringing with them problems of supply as well as problems of waste disposal. The ever-increasing demand for energy has stimulated plans for the development of additional hydroelectric power. The need for moving bulk commodities at low cost has furthered the development of modern navigation facilities. Each such development has required a portion of the total water supply. At times the same portion or source has been wanted for more than one use, and conflicts have developed. Although as yet minor, these conflicts warn of other greater conflicts in the future.

In 1900, Alabama was an agricultural State, and her demands for water were modest and evenly distributed. By 1955 the greater part of her population and industries were concentrated in a few areas. This change carried with it a paralleling concentration of demand for water—a demand which frequently exceeded the natural supply of the immediate area. Mobile is an example. A few years ago Mobile changed its source of supply from local small creeks to a large reservoir several miles from the city. Another example are the new locks being constructed on the Tombigbee, which are larger than the old ones, and require more water to fill them. Many other instances could be cited. As the State grows it makes greater demands on the supply of water, and locally the demands may at times exceed the supply that nature has provided.

Alabama is unusually blessed with water resources. On an average, Alabama streams discharge 50,000 million gallons daily. In dry years the amount is smaller than this and during periods of extreme droughts the supply is only one-thirtieth of the average. The water available in an extreme drought period is all that can be claimed as a natural dependable surface supply, but by storing water in wet seasons, the dry-season supply can be increased. With enough such storage the supply is theoretically as large as the average discharge, which in Alabama is many times larger than the predicted uses for years to come. Mobile's reservoir on Big Creek is a step toward using the average discharge of that creek.

The answer to the problems of supply faced in these recent dry years and for the future lies in making greater use of water drawn from storage. Part can come from the supply that nature stores in the ground. Part can come from water stored behind dams. By making sufficient use of storage, supplies can be made many times larger than any foreseeable use.



Progressive Five-Year Averages, Tallapoosa River at Wadley, Alabama

Journal of the
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Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1460)

Note: Paper 1460 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 5, December, 1957.

A SUPPLEMENTAL NOTE ON VALUATION AND DEPRECIATION[a]

Discussion by Maurice R. Scharff

MAURICE R. SCHARFF,[1] M. ASCE.—The writer is grateful to Messrs. Jeynes, Blewitt, Bary, and Brown for contributing to the discussion of this important subject and thus fulfilling his expressed hope of useful results for the profession. He shares fully with Mr. Jeynes the opinion that it is surprising that the literature on the subject is not more extensive.

The author appreciates particularly the helpful suggestions of Mr. Jeynes regarding the extension of the author's oversimplified formulae so as to give effect to deferment of federal income taxes as a result of differences between book depreciation and one of the alternative methods of calculating accelerated depreciation for tax purposes available under existing laws. It would be helpful if further studies could be made and published to show the effect of other permissible alternative methods of calculating depreciation for tax purposes.

Mr. Blewitt has also called attention to interesting opportunities to extend the usefulness of the author's analyses so as to give effect, not only to these permissible variations in depreciation for tax purposes, but also to the effect of retirement dispersion, and to the effect of prospective continuing progress in the arts in connection with the earlier freedom of choice as to type of replacement in the case of an existing, as compared with an alternative new facility. The writer agrees as to the usefulness of both suggestions, and hopes that further studies may be carried out and published for use by students in this field.

The author has studied attentively the interesting paper by Messrs. Bary and Brown, to which they refer in their discussion, as well as the published discussions of that paper including the closing discussion by them. He is not certain that he understands completely, even after such study, all the implications of their extensive mathematical analyses. However, as far as he does understand them, he is not convinced that they have "clearly demonstrated," as they contend, that the "true discount rate applicable to all present worth computations" is (R-IiB) instead of R, as used in the author's paper. It seems to the author that the writers have confused the issue by introducing a concept of "money source," which is related to an entirely separate subject of financial judgment as to creation of debt and reserves, leverage and distribution of net income, and which is irrelevant to the rate of return after all taxes, including income taxes, on alternative capital investments. If this view is correct, then the "true discount rate applicable to all present worth computations" is R, and not (R-IiB).

a. Proc. Paper 1184, February, 1957, by Maurice R. Scharff.
1. Cons. Engr., New York, N. Y.

The author commends Messrs. Bary and Brown for the energy with which they have supported their thesis in the paper and discussions to which they have referred, and hopes that they will continue to write and publish on the subject so that students in this field, including the writer, may finally be sure that they understand fully the writers' contention.

ARCH DAMS: STRESS STUDIES FOR ROSS AND DIABLO DAMS[a]

Discussion by William T. Moody

WILLIAM T. MOODY.[1]—The authors have contributed measurably to the store of recorded information which should aid engineers in correlating design criteria with field measurements of structural action. True economy of design, which is an engineer's primary objective, can be improved only with advancement of understanding of the behavior of structures under load.

The writer wishes to point out the physical significance of certain terms in the fundamental equation for determining the creep parameters. This significance suggests a different notation which it is believed will promote clearer understanding of the creep phenomenon as expressed by the author's equation. Their relation is given as

$$\varepsilon = \frac{1}{E} + F(K)\ \ln(t+1). \tag{1}$$

If $t = 0$ in (1), E is immediately seen to be an instantaneous modulus of elasticity, i.e., the constant of proportionality between unit stress and the instantaneous strain at initial loading. The nature of the "creep function," F(K), is obscured by its notation. However, by placing $t = e-1$, where e represents the base of natural logarithms, (1) becomes

$$\varepsilon = \frac{1}{E} + F(K). \tag{2}$$

F(K) is now seen as the reciprocal of a modulus of elasticity. Furthermore, this reciprocal is the constant of proportionality between unit stress and the strain that occurs in the first e-1 days after the initial deformation.

These and other considerations suggest rewriting (1) as

$$\frac{\varepsilon}{\sigma} = \frac{1}{E_0} + \frac{1}{E_1}\ \ln(1+kt), \tag{3}$$

in which

ε represents elastic plus creep strain [0] when subjected to

σ a constant stress $[FL^{-2}]$,

E_0 instantaneous modulus of elasticity—the constant of proportionality between σ and the instantaneous strain at initial loading $[FL^{-2}]$,

E_1 creep modulus of elasticity—the constant of proportionality between σ and the strain which occurs in the first e-1 days after initial deformation $[FL^{-2}]$,

a. Proc. Paper 1267, June 1957, by Joe T. Richardson and Owen J. Olsen.

1. Engr., U. S. Dept. of the Interior, Bureau of Reclamation, Denver Colorado.

t time after loading [T],

k a constant whose value depends upon the unit of measurement of t; k = 1 when t is measured in days $[T^{-1}]$.

In the notation above, dimensions of each symbol are indicated in terms of force, F, length, L, and time, T, in brackets as a part of the definition.

Equation (3) is seen to be dimensionally correct while at the same time it conforms with accepted notation used in theory of elasticity.

Addition of the k in the term under the natural logarithm is necessary since the argument of such functions must be a dimensionless quantity.

A final simplification may be made, if desired, by letting $E_0 = nE_1$ so that

$$\varepsilon = \frac{\sigma}{E_o}\left[1 + n \ln(1 + kt)\right]. \tag{4}$$

While notation sometimes seems to be unimportant, it has been amply demonstrated in such cases as the Arabic vs. Roman number systems and in Leibniz' vs. Newton's notations for the calculus that a judicious choice of symbols can considerably increase the utility and clarity of a mathematical development.

MULTI-LAYER PENSTOCKS AND HIGH PRESSURE WYES[a]

Discussion by Carlos S. Ospina

CARLOS S. OSPINA,[1] M. ASCE—The writer has two questions in regard to this interesting paper:

a) Vent hole in multi-layer construction.

It is not clear to the writer why these vent holes are needed, a fuller explanation by the author would be appreciated.

b) Cubatao underground Plant Manifold.

The writer understands that this manifold failed during testing. As failures are such valuable teachers, it might be very helpful if the author were to describe the failure and the steps taken to repair it and avoid its repetition.

a. Proc. Paper 1344, August, 1957, by Ewald Schmitz.
1. Partner, OLAP, Bogota, Columbia.

BIBLIOGRAPHY: UNDERGROUND HYDROELECTRIC POWER PLANTS[a]

Discussion by Carlos S. Ospina

CARLOS S. OSPINA,[1] M. ASCE.—The writer wants to add one important item to the Bibliography.

Under Section A, item 2 d one should mention that further discussion of Dr. Jaeger's paper appeared in the Proceedings, Institution of Civil Engineers, V. 4 pt. 1, July 1955, P. 545-596. The authors of the discussion were:

F. H. Knapp, J. F. Hagrup, Jean Pelletier, C. G. Chatham, Joseph Talobre, A. Terrisse, F. L. Lawton, A. J. Harris, G. T. Colebatch, A. Hutter and J. Hanimann, Samuel Judd, H. K. Brickey, F. H. Lippold, Otto Frey-Baer, O. M. Ferraz and Charles Jaeger.

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a. Proc. Paper 1350, August, 1957, by J. Barry Cooke and Arthur G. Strassburger.

1. Partner, OLAP, Bogota, Columbia.

Paper 1488 December, 1957 PO 6

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CIRCULATING WATER SYSTEMS OF STEAM POWER PLANTS[a]

R. T. Richards,[1] A.M. ASCE
(Proc. Paper 1488)

SYNOPSIS

Circulating water systems for condenser cooling at large steam power plants are major water-pumping facilities which often move enough water to supply the domestic needs for a city of over a million and a half people. The need for reliability and the possible savings in installation and operating costs justify careful hydraulic design. This paper discusses some of the system hydraulic features which require special attention. Among these are the economics of pumping, the calculation of pumping head, the hydraulic design of pump suction chambers, maximum siphon recovery, air binding and water hammer.

INTRODUCTION

The circulating water systems for condensing steam at the turbine exhausts of steam power plants require very large and often complex water-pumping installations. The desirability of careful mechanical and hydraulic design becomes apparent when one considers the very large quantities of water which must be moved through a large condenser. A water use of 1-1/2 cubic feet per second per 1,000 kw will result in a total requirement of 300 cfs for a 200,000-kw unit. This is 135,000 gallons per minute or, in terms of water works practice, 194 million gallons per day, enough water to supply domestic and industrial needs for a city of over a million and a half people.

There are many factors which the designer must consider if he is to provide such quantities at the least possible cost and with the least difficulty in

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a. Paper presented at a meeting of the American Society of Civil Engineers, New York, N. Y., October, 1957.

1. Civ. Engr., Ebasco Services Inc., New York, N. Y.

operation. Safety must also be considered because of potentially serious water hammer surges which may occur in some systems.

The purpose of this paper is to highlight some of the more important elements which must be considered in a well designed circulating water system.

For convenience of discussion the major hydraulic considerations in circulating water system design may be set forth as follows:

1) Economics of pumping.
2) The circulating water system layout, including conduit plan and profile, location of intake and discharge, design of hydraulic shapes to minimize head losses.
3) The selection of pumps, with particular attention to the determination of the proper design pumping head, and to pump operation at heads below and above the design point.
4) The pump structure and the proper guidance of flow to the pumps, especially critical for suction chambers supplying vertical pumps.
5) The setting of the seal well or final discharge point to obtain maximum siphon recovery.
6) Air binding and problems of conduits under vacuum.
7) Water hammer possibilities.

The Economics of Pumping

The economics of pumping water is one of the first considerations in the design of the circulating water system. The design details are influenced by several cost relationships, among which are the following:

a) The investment costs of the pump and motor increase as the head increases.
b) As the conduit size is decreased the construction cost also decreases, but because of the resulting increase in friction and in total pumping head the pumping costs rise.
c) When refinements of water passages are being considered pumping costs will usually be decreased as the cost of making the refinements is increased.

Taking these factors into consideration we have some of the yardsticks by which we can measure the desirability of a particular circulating water system design.

As an example, let us determine the proper size for a 3,000-foot length of straight pipe carrying a fixed quantity of water continuously throughout the year. A chart can be prepared wherein the costs of purchasing and installing the pipe are added to the costs of pumping the water and providing the extra power capacity in the power plant ("demand" cost). The most economical pipe size gives the minimum sum of these factors. However, the simplicity of this calculation is often complicated by the variation of conduit flow during the year or the variation over a number of years as more units are added to the system. Neither the size nor installation date of future units is available in many cases.

The cost of moving a large quantity of water against 1 foot of head may well be such that considerable expense is justified in making a design refinement which would reduce this pumping head. Consider a typical 200,000-kw unit using 300 cfs. With a fuel cost of 20 cents per 1,000,000 Btu and with continuously operating pumps the capitalized cost per foot of head would be about

$ 5,000. To this add a demand cost of $2,600, giving a total of $7,600 which is a rough measure of what the designer can afford to spend to save one foot of head.

Fig. 1 is a chart from which these approximate cost figures were obtained. This chart was prepared by the author and originally published in Power Magazine February, 1956. It permits the rapid determination of capital costs, annual pumping costs and "demand" costs per foot of head. Starting at the upper left-hand side of the chart with a known water quantity, one proceeds as indicated by the dotted lines. The demand costs are read from the top of the chart, the annual and capital costs from the right and left sides respectively at the bottom.

Circulating Water System Layout

The most suitable circulating water system has an intake located on a lake, river or tidal area in such a manner as to obtain the coolest, clearest water and a discharge located to prevent warm water recirculation (except as specifically provided for in cold climates). Such location is a major study in itself and is not within the scope of this paper. The author is concerned here with designing the most efficient conduit system to move water between predetermined points of intake and discharge.

The major concentrations of head loss are usually at three points in the system: at the pump discharges, at the piping entering and leaving the condenser, and at the final discharge structure. At all these points numerous changes in direction and velocities occur.

Every effort should be made to reduce the number of abrupt transitions, bends and obstructions within the system. It is also worth while to consider the use of diffusers at the pump discharge, at the outlet of the condenser piping into the main conduit, and at the final discharge. A diffuser is a gradual increase in the water passage area which permits recovery of some of the velocity head. At bends a vaned elbow, common in wind tunnel work and discussed below for pump suctions, may be a worthwhile head saving device where lack of space requires sharp bends or a series of bends in different directions. Whether the use of diffuser, vaned elbows or other refinements is justified is a matter of economics—the cost of the refinement compared with the expected reduction in cost of pumping.

Pump Design Operating Heads

The determination of the proper design head for a circulating water pump requires consideration of many factors. The friction coefficients in large size conduit are not the same as those customarily used for small (12" and less) water piping. They are usually much smaller in large conduits and therefore result in less friction loss. Also, the intake water level at many plants varies over a wide range, as at river and tidal sites. The selection of the proper water level for pump head design requires a careful study of seasonal variations. It is not usual practice to design the pumps for the extreme low-water condition.

The intake water level assumed for pumping head calculations should be carefully considered so that the pump will operate as close to its design point as possible over the majority of its operating life. Using as an example a

typical quantity-head curve for a mixed flow pump it may be seen from Fig. 2 that if the intake water level should drop such that the original calculated head was exceeded by 20% the actual loss of flow would be only 6-1/2%, possibly not serious if occurring at rare intervals. It must be kept in mind, however, that extreme low water should be used when determining the depth of the intake structure and the length of vertical pump to insure that the pumps can draw water at all times.

The author's experience and information gained from the major pump manufacturers clearly indicate that the underdesigned pump is rare and that overdesigned pumps are the rule. The result is not conservative pump design, but a design that is wasteful of power and may result in premature failure of the pumps. As the pump operates farther out on its quantity-head curve it requires a greater submergence which is not always available. The result may be cavitation at the impellers with resulting destructive pitting. Even with ample submergence the pump may operate roughly at extreme runouts.

Submergence is defined as the depth of water over the pump impeller or, as more commonly specified, over the bottom of the bell. Pump manufacturers should be requested to provide not only a recommended submergence at design point, but a curve of required submergence covering the entire operating range of the pumps.

With pump heads properly determined there still may be operating procedures which will unduly strain the pumps. Operating a reduced number of pumps through all condenser shells on the system may result in an undesirable pump runout. This should be investigated in design and the operators alerted to its undesirable effects.

The Pump Structure

Pumps are sensitive to the flow patterns at their impellers. Horizontal and vertical centrifugal pumps generally present only minor problems since a length of suction pipe will often properly distribute the flow. But with vertical axial and mixed flow pumps the problem is acute. Even with the best of care in designing the flow approach, vortices may form and draw air into the system. Rough and often damaging pump operation results and air may also cause pitting problems in condenser tubes. In a poorly designed intake general hydraulic unbalance at the impellers may result in rough operation, pitting due to cavitation and excessive bearing wear even if no vortices are present.

The designer must rely on his own experience, model studies and the recommendations of the pump manufacturer in laying out the structure. Unfortunately there is a wide divergence of opinion among the major manufacturers of large vertical pumps as to what constitutes a proper suction chamber. Where construction space permits, the manufacturers recommend a structure with straight sides (for a distance of say 10 pump bell diameters) and a horizontal bottom. But recommendations for side, bottom and back clearances for the pumps vary widely for similar pumps. The author has investigated irritating flow problems in chambers approved in detail by the manufacturer and therefore assumed to be in accordance with best practice.

There have been many attempts to insure balanced flow to the pump impellers by specifying various types of baffles, suction tubes and turning vanes but these have not proven to be cure-alls.

At several recent installations the author's firm has used a vaned elbow

under the pump bell, consisting of a set of five to eight horizontal turning vanes, which guides the flow up into the pump. A typical elbow is shown in Fig. 3. Vanes are made of carbon steel or sometimes a metal less subject to corrosion. Six major plants now in operation and several under construction have pumps so equipped. The elbow permits the use of a pump bay only a few inches wider than the pump bell itself (as compared with two pump bell diameters normally recommended). To offset this advantage, however, the chamber must be deeper. Hydraulic performance has been satisfactory, both in new installations and in those where the elbows have been installed to correct existing poor hydraulic conditions. The vaned elbow is not a solution for all intake design problems, but it has proven its value in many cases and is approved by at least two of the major circulating water pump manufacturers.

Where pumps cannot be installed in the ideal straight-sided chamber, as in many additions to old plants, there is no substitute for model tests to determine the proper design for hydraulic passages.

Maximum Siphon Recovery

For all but cooling tower installations the final discharge point for the circulating water system will be well below the condenser, resulting in a section of the system operating as a siphon under less than atmospheric pressure.

If the plant is high on a lake shore or river bank, the lower the discharge point is set, the lower will be the pumping head as the static head is reduced. The limit to reduction of pumping head is the maximum siphon or amount of vacuum which can be permitted in the system.

The measure of vacuum is the vertical distance from the hydraulic gradient up to any given point within the watter passages. The maximum vacuum is therefore not a fixed figure representing the distance from the discharge water surface up to the highest point in the circulating water system, but is a figure which varies with friction in the discharge conduit. If discharge friction is high, the condenser may be set higher or the system discharge lower to utilize maximum permissible siphon recovery. Fig. 4 indicates a typical relationship between the conduit and the gradient.

At sea level theoretical maximum vacuum is about 34 feet of water and vapor pressure for 97° F water temperature is 2.0 feet; therefore, we could theoretically take advantage of a minus 32-foot pressure to reduce pumping head. In practice there are two limitations, however, to how much of this figure we may actually use to advantage. First, if the conduit must be evacuated by vacuum pumps the rate at which commerically available pumps can remove air falls off rapidly at high vacuums, say 28 feet or 29 feet at sea level. Secondly, the rate of air release from the water increases rapidly as the vacuum is increased. There will come a point where air is released faster than it can be removed by vacuum pumps or by the natural pumping action of turbulent water. Air then accumulates and may block off upper condenser tubes, reducing the heat exchanging capacity of the condenser. It may also cause air binding in other sections of the conduit. The author is familiar with one station which successfully maintains a siphon between 29 feet and 30 feet. For design purposes a maximum of 27 feet to 28 feet at sea level is recommended. For plants at higher elevations this siphon must be reduced by the amount of reduction of atmospheric pressure.

For determining probable pump head, the designer must use a reasonable

figure for siphon recovery; however, the seal well should be so designed that the operators are not limited to the maximum value of siphon used for the pump design. The permanent crest of the seal well weir, which controls the siphon, should be set low enough so that maximum possible siphon can be recovered at full flow. The operators can then make stop log adjustments to give optimum performance at any flow.

Fig. 4 shows that with a two-pass condenser a substantial reduction in pumping head can be achieved by using top inlet and bottom outlet, as indicated in the dotted lines. The physical top of the outlet water box may be lowered from Point A to Point A^1 (as much as 1/2 the height of the condenser depending upon interior condenser design); the hydraulic gradient remains unchanged. With this arrangement the seal well can then be lowered to take advantage of maximum permissible siphon, thus lowering the entire gradient and the pumping head.

Note that the siphon is used only to regain pumping head. It is objectionable in itself and when no pumping head advantage can be gained (as at many water level plants) it should be kept as low as possible to decrease potential air-binding problems.

Air Binding and Conduits Under Vacuum

The greatest potential source of excess head loss is air binding. Air may collect at high points or, often far more serious, water passages may fail to flow full on downward slopes.

Many condenser discharge conduits are designed to operate under a partial vacuum, not only in the immediate vicinity of the condenser, but throughout a large part, if not all, of the conduit. Even the condenser inlet conduits may operate in this manner. It is in these conduits under partial vacuum that serious air-binding problems occur.

Generally where a conduit under vacuum slopes downward, as opposed to a vertical drop, it will not flow full unless air is continuously evacuated; the head loss will be nearly equal to the fall of the pipe (i.e., an open channel flow condition under less than atmospheric pressure). This can be the source of substantial loss. The author has in mind two plants where the excess head losses due to this condition were 15 feet and 19 feet or 35% and 50% above the total head for which the pumps were designed. At these two stations and at several others where the design would lead to a similar condition continuous air evacuation has eliminated the problem.

There are two methods for changing pipe elevation: slope the pipe or drop it vertically. The vertical drop is usually self-priming because of the air-pumping action of the bend turbulence. However, a vertical drop introduces a considerable head loss in two reverse 90 degree bends, and also must be anchored securely even under low head to prevent movement under possible water hammer thrusts. The author usually favors a sloping design for large conduits. The conduit must be tapped for air evacuation not only at the top but along the slope and appropriate air removal equipment provided. Vacuum pumps and water and steam operated air ejectors have been used. These evacuation systems may require power, controls and a source of clean seal water (for vacuum pumps), all of which must be considered when determining the most economical method of making the change in grade.

Within the power plant structure a sloping pipe leaving the condenser may

possibly be self-priming if the general turbulence in the pipe is great enough (because of one or more preceding bends); however, if the designer cannot avoid the slope, he should provide air evacuation taps along the slope and be prepared to recommend possible future continuous evacuation.

Collection of air along slopes under pressure does not appear to be serious; small taps at the top and a few feet below the top of the slope can be hand operated to release air on initial start-up. This will speed up the complete filling of the system.

Water Hammer

Generally speaking the possibilities of serious water hammer surges increase with the length of the circulating water system. Surges may occur on pump start-up, on valve closures or following pump trip-outs. Simultaneous tripout of all pumps operating on the circulating water system is usually the most serious source of excess surges.

Following tripout, the water leaving the condenser (or other high point on the system) discharges at a faster rate than it can be supplied by the rapidly decelerating pumps. The result may be a separation of the water column leaving a vacuous space. When the discharging water exhausts its forward momentum it will reverse its direction of flow and close up this space. At the instant of closure a sharp surge will occur having a magnitude of as much as 100 times the reverse velocity destroyed in feet per second. With discharge lines several thousand feet in length this surge may reach two or three hundred feet of head, many times higher than the normal design head.

Vacuum breakers, air chambers, open manholes and other devices may be used to control water hammer. Fortunately, only a small percentage of stations will require special water hammer treatment, but its consideration in design should never be neglected.

Several actual examples of the development of surges, a review of the elements involved and the methods of surge control are discussed in the author's paper "Water Column Separation in Pump Discharge Lines" which appeared in the August, 1956 ASME Transactions. Full tripout tests on the circulating water system of the Ninemile Point Station of the Louisiana Power & Light Company clearly revealed the inherent possibilities of water hammer at a typical large station.

CONCLUSION

Improvements in many of the design features discussed above have developed from observation of operating circulating water systems. The accuracy of pump head calculations, the hydraulic performance of a given pump intake, air-binding problems and the occurrence of water hammer can only be verified by field observation. The cooperation of system operators is therefore most desirable. Every effort should be made to observe and report on actual system operation, especially where difficult hydraulic problems have been encountered in design.

It is the author's hope that this brief review of the hydraulic features of circulating water system design will not only be of interest to power plant design engineers, but that it will also stimulate discussion leading to our better understanding of the problems involved.

COST OF PUMPING WATER THROUGH ONE FOOT HEAD

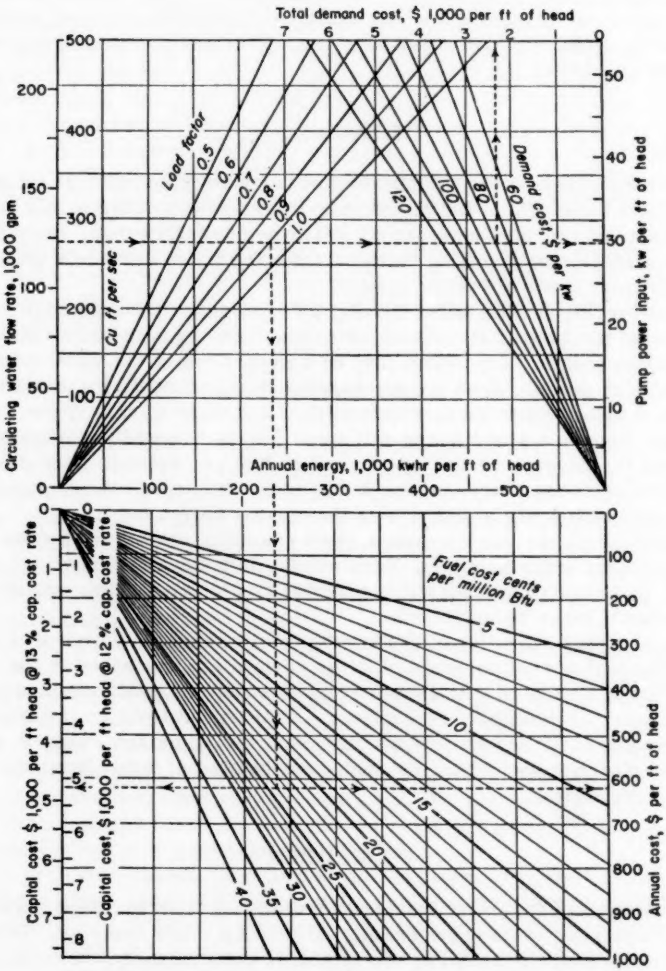
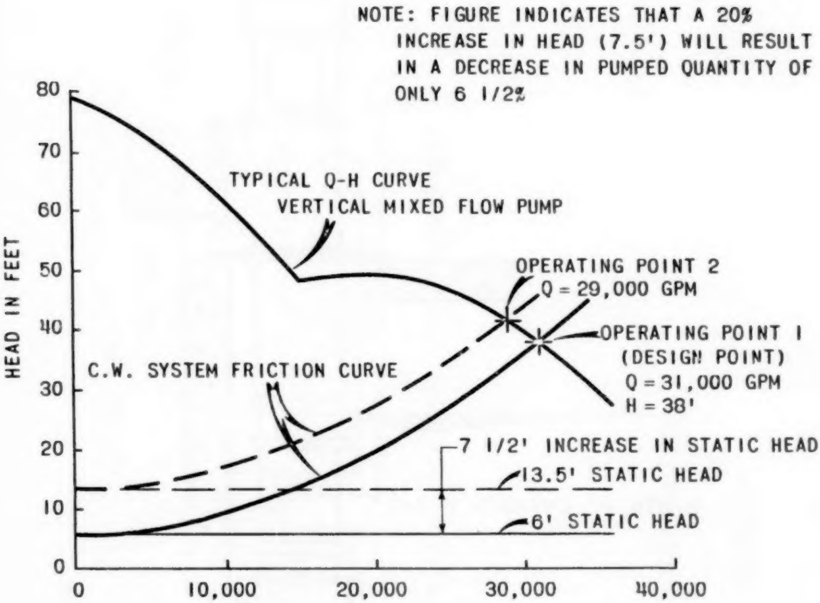


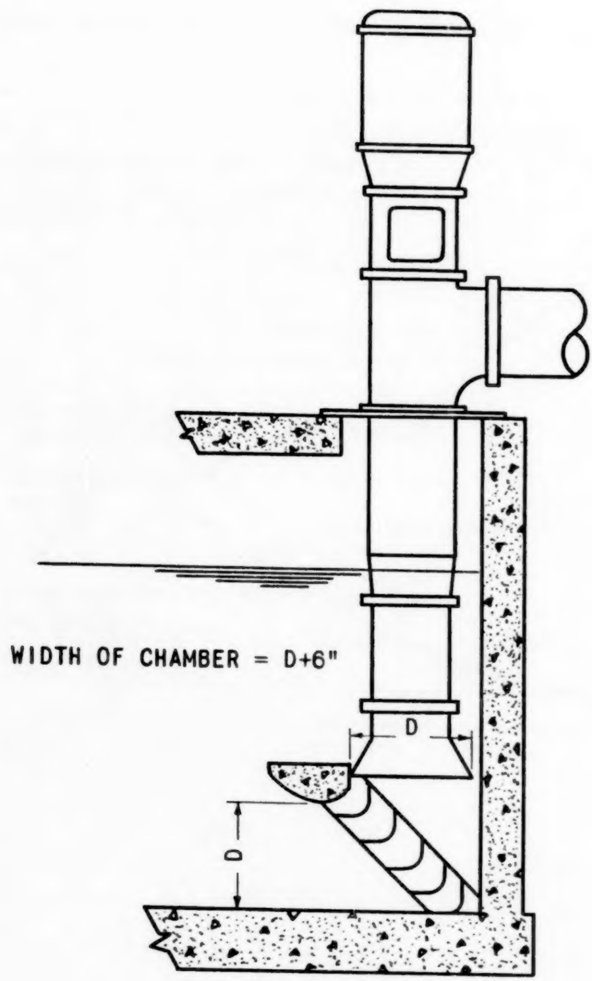
Chart is Based on: 78.3% over-all pump and motor efficiency, 8760 hr per year, 11,000 btu per kwhr heat rate.

FIGURE 1



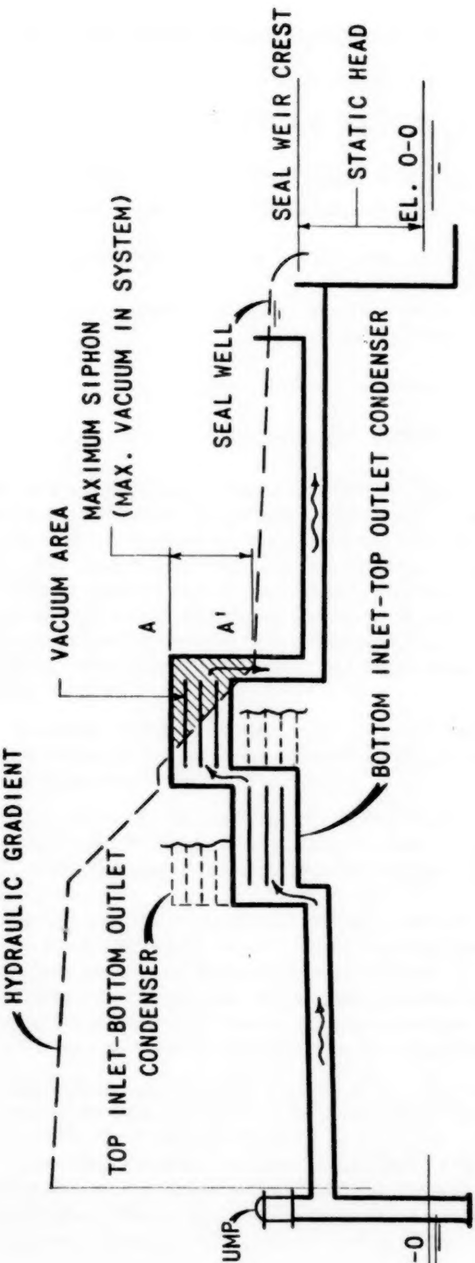
QUANTITY - GPM

FIGURE 2



VANED ELBOW PUMP SETTING

FIGURE 3



C.W. SYSTEM SIPHON

FIGURE 4

Paper 1489 December, 1957 PO 6

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COMPUTER STUDIES OF PENSTOCK AND GOVERNOR SYSTEMS

Eldo C. Koenig[1] and Howard A. Knudtson[2]
(Proc. Paper 1489)

INTRODUCTION

The overall design of a modern hydro-electric generating plant dictates the design of its individual components—reservoir, penstock, surge tank, governor and turbine generator so as to provide optimum operation of the total system. In a system analysis such as this, an Electronic Differential Analyzer is a very effective tool in that the nonlinear differential equations describing the system may be easily simulated and various parameters may easily be varied over wide ranges to note the effects on system performance. This discussion proposes to demonstrate the use of the differential analyzer in the following hydraulic studies:

1. Penstock and differential surge tank for the condition of load rejection.
2. Governor study including effects of the penstock with elastic walls, simple surge tank, and turbine.

For the study of the penstock and differential surge tank, the analysis includes the nonlinearities as well as spill-over of the riser into the outer tank and the resulting computer solutions are compared with test results of an actual system.

Computer studies of governor systems have been made before;(1, 2) however, in these studies, it was assumed that the penstock walls were inelastic and frictionless and no surge tank was present. It was felt that these factors may be important in a study and should be investigated. To reduce the complexity of the problem of such a complete system, the nonlinear differential equations for the turbine and penstock are linearized.

Note: Discussion open until May 1, 1958. Paper 1489 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 5, December, 1957.

1. Engineering Analysis Section, Allis-Chalmers Mfg. Co., Milwaukee, Wisc.
2. Formerly of the Engineering Analysis Section, Allis-Chalmers Mfg. Co., Milwaukee, Wisc., now employed in the Aeronautical Div., Minneapolis-Honeywell Regulator Co., Minneapolis, Minn.

Nomenclature

a, b, d, e, f	Floating lever lengths of governor system
A_c	Area of conduit, sq. ft.
A_a	Port area between riser and outer tank of differential surge tank
A_r	Riser area of differential surge tank, sq. ft.
A_t	Tank area of simple surge tank or area of outside tank of a differential surge tank, sq. ft.
C	Coefficient of discharge through an orifice
C_e	Area of equivalent surge tank for lumped parameter representation of elastic penstock walls, sq. ft.
C_r	Coefficient of discharge through the orifice between riser and tank of differential surge tank
E	Modulus of elasticity, lbs. per sq. ft.
F(hg)	Spill-over from riser to tank as a function of gate head, cu. ft. per sec.
g	Instantaneous gate area, sq. ft.
g_r	Acceleration of gravity, ft. per sec.$^{(2)}$
h_g	Instantaneous tank or gate head, ft.
h'	Supply head, ft.
h_t	Instantaneous head in outside tank of differential surge tank, ft.
J	Inertia constant of turbine and rotating members, lbs. ft. per sec.$^{(2)}$
K_4	Speed per unit displacement of flyball rod
K_3	Velocity of piston in regulating cylinder per unit pilot valve displacement
K_c	Friction coefficient of conduit
K_1	Spring constant
K_2	Coefficient of viscous damping of dashpot
K	Constant
l_d	Instantaneous load of turbine
L_c	Length of penstock, ft.
L_r	Riser height of differential surge tank
n	Instantaneous speed of turbine, radians per sec. or the exponent used in defining flow through an orifice whose value is a function of the type of orifice
P	Pressure, lbs. per sq. ft.

q_a	Instantaneous flow through orifice between riser and tank of differential surge tank, cu. ft. per sec.
q_c	Instantaneous flow through conduit, cu. ft. per sec.
q_g	Instantaneous flow through gate, cu. ft. per sec.
q_r	Instantaneous flow through riser or center tank of differential surge tank, cu. ft. per sec.
r_c	Radius of conduit, ft.
T	Wall thickness of conduit, ft.
y	Flyball rod displacement
x_1	Displacement of spring
x_2	Pilot valve displacement
x_4	Displacement of dashpot
ρ	Water density, lbs. per cu. ft.

Instantaneous quantities written as sum of steady state and fluctuating values:

$$h_g = H_{go} + H_g$$

$$h' = H'_o$$

$$g = G_o + G$$

$$n = N_o + N$$

$$l_d = L_{do} + L_d$$

$$q_c = Q_{co} + Q_c$$

Penstock and Differential Surge Tank

The action of a differential surge tank in a hydraulic system is quite superior to that of a simple tank of the same size. To obtain comparable performance, the simple tank would need to be considerably larger. The functions of a surge tank are to supply or absorb the sudden changes in flow required by the turbine upon load changes and to rapidly damp out the resulting oscillations in water level. These actions are effectively executed by a correctly designed differential surge tank. The diagram of the complete installation is shown in Fig. 1. The outer tank performs the water-storage function and damping is produced primarily by the high velocity flow through the port between the inner and outer tank. Also, the best results are obtained by placing the surge tank as close to the turbine as is practicable.

Equations 2, 7, and 8 of Appendix I describe the system of Fig. 1, and the computer circuit for solving the equations is shown in Fig. 3. The non-linearities which were included in the computer circuit are as follows:

1. The friction head drop in the conduit and riser is proportional to the square of the flow.
2. The flow through the riser port is a non-linear function of the pressure across it and the port area.
3. The height of the riser is such that for large load rejections, the water will overflow from the riser into the outer tank.

Details of the computer setup are discussed in Appendix II.

Results shown in Fig. 4 were obtained for the following system parameters:

$L_c = 3400$ ft.

$L_r = 237$ ft.

$A_c = 177$ sq. ft.

$A_t = 662$ sq. ft.

$A_r = 143$ sq. ft.

$A_a = 8.4$ sq. ft.

$K_c = 2.27 \times 10^{-6}$

$n = 0.7$ (exponent, used in defining flow through port)

$C_r(2gr)^{0.7}n = 1$

The solutions are voltage functions of time obtained by a computer with repetitive type operation which means, they are measured on an oscilloscope. Fig. 4a shows the photograph of the oscilloscope trace of flow through the gate, qg, and was the driving function of the computer circuit of Fig. 3. The resulting flow through the conduit, q_c, is shown in Fig. 4b. The gate head, h_g, and the tank head, h_t, obtained from the computer are shown in Fig. 4c by the solid lines and the broken lines show results from tests. The coefficient of friction of the conduit, K_c, was adjusted in the computer to give a ten foot difference in head at the tank between the steady-state full-load and steady-state no-load conditions. In general, the computer results agree fairly well with the results from tests. Probably the difference is chiefly due to the characteristics of the port between the riser and tank. Also, the inertia of the water in the riser above the port (a variable inertia) is neglected.

The differential analyzer solution of the problem has a definite advantage over other methods of solution in that the system parameters may be readily varied to show the effects on system performance immediately. Thus, for general information, h_t and h_h are shown in Fig. 5 as solutions resulting from variations in certain system parameters as follows:

5a	conduit length increased 20 per cent
5b	conduit length decreased 20 per cent
5c	port area increased 20 per cent
5d	port area decreased 20 per cent
5e	no limit on riser height
5f	no time lag on spillover; i.e., water from spillover immediately adds to water in tank
5g	exponent of head drop through port 0.6
5h	exponent of head drop through port 0.8

Complete Hydraulic System

This section of the analysis is concerned with a complete hydraulic system, namely, the reservoir, penstock, surge tank, governor, and turbine. The effect of conduit elasticity is included in a lumped parameter sense. When the effect of the penstock system is included in a governor study, the small changes

in speed and gate movement relative to steady state values permit simplifications in the equations which are not permissible when studies are made for full load rejection. The non-linear equations are linearized in this simplifying procedure and thus solutions are more easily obtained. The linearizing of the equations is given in Appendix I and the resulting equations 29, 30, and 31 are the equations of the penstock system used in the governor study.

The hydraulic governor system chosen for consideration in the study is shown in Fig. 7. The functioning of the governor mechanism has been analyzed in reference 1 and equations 34, 37, and 38 of Appendix I are the resulting equations. These three equations and the linearized equations 29, 30, and 31 of the penstock system were used in constructing the computer circuit of Fig. 8 and apply to a complete system with the following principal assumptions:

1. A single hydroelectric unit supplies the load.
2. The generator supplies real power with instantaneous voltage regulation so that the power is independent of speed and the torque varies inversely with speed.
3. The turbine efficiency is constant for small changes in speed, head, and gate opening.

Equation 34 is non-linear if the pilot valve of the governor has valve lap and/or is limited in its motion. The computer circuit of Fig. 8 shows how those non-linearities may be included by means of dead-zone and bounding components.

Results were obtained from the computer for a system with the following numerical values:

$\alpha = 100$	$\frac{Q_{co}}{G_o} = 11.3$
$T_c = 58$	$\frac{L_{do}}{G_o} = 2.0 \times 10^5$
$\beta = 0.29$	$\frac{L_{do}}{H_{go}} = 1.20 \times 10^5$
$T_m = 12 \times 10^6$	$H_{go} = 290$
$\frac{a+b}{aK_4} = 1$	$G_o = 177$
$K_3 = 1$	$\frac{K_2 be(d+e)}{K_1 ad^2} = 4$
$N_o = 12$	$\frac{K_1 e^2}{K_2 d^2}$
$Q_{co} = 2000$	

Equation 34 was considered linear in the study; i.e., there was no overlap and limited motion of the pilot valve corresponding to zero dead-zone and no limit for the computer components. An instantaneous load change was applied on the system and the quantity $T_{te} = 0.29(A_t+C_e)$ was considered the independent variable in obtaining the results shown in Fig. 9. The most interesting results were obtained when the sum of the tank area and the elastic constant of the conduit walls, $(A_t + C_e)$, was small. Fig. 9 shows speed, N, gate head, H_g, penstock flow, Q_c, and gate area, G, as a function of time for values of T_{te}

equal 1.0, 0.1, and 0.01. It is noted that the worse conditions exist for all these solutions when T_{te} equals 0.1. It therefore appears that for a given system there may be values of T_{te} which should be avoided and since these values are likely to be small, these undesirable conditions could exist when only the surge tank effect of the elastic walls is present. Corrective measures could perhaps be made in the governor and there is no better means for making these explorations than with a computer.

The values of the fluctuating quantities obtained as solutions in Fig. 9 are seen to be small compared to the given steady state values so that the assumptions made in Appendix I for linearizing the equations are valid.

CONCLUSIONS

The non-linear differential equations describing a penstock with elastic walls and a differential surge tank are easily solved with an Electronic Differential Analyzer and parameters are readily varied on the computer to study a system. Solutions obtained by the computer check reasonably well with results obtained from tests.

A governor study was made with a Differential Analyzer which included the penstock with elastic walls and a simple surge tank. The assumptions made in Appendix I for linearizing the equations were found to be valid. Results for the system under study showed that it was important to consider the effect of the elastic walls of the penstock in obtaining an optimum governing system.

The authors wish to express their appreciation to Mr. B. R. Nichols of the Hydraulic Dept., Allis-Chalmers Mfg. Co. for supplying the numerical data and for suggestions made during the computer studies.

APPENDIX I

A Penstock System with a Differential Surge Tank

A sketch of a penstock with a differential surge tank is shown in Fig. 1. In analyzing the penstock for the effects of elastic walls, lumped parameters are considered and the equivalent penstock is shown in Fig. 2. Here the effect of the elastic walls is simulated by surge tanks concentrated on the ends of the conduit which is now considered to have rigid walls. The flow to each of these assumed tanks is

$$q_e = \left(1 + \frac{r_c \rho h}{2ET}\right) 2\pi r_c^3 \frac{\rho}{ET} \frac{L_c}{2} \frac{dh}{dt} \cong C_e \frac{dh}{dt} \tag{1}$$

where

$$C_e = 2\pi r_c^3 \frac{\rho}{ET} \frac{L_c}{2}$$

and h is the head of the assumed tanks.

The equations which describe the conditions of the conduit and the surge tank are

$$(h' - h_g) = \frac{L_c}{A_c g_r} \frac{dq_c}{dt} + K_c |q_c| q_c \tag{2}$$

$$q_c - q_g = q_e + q_r - q_a + F(h_g) \tag{3}$$

where F(hg) represents flow over the riser, or center tank, as a function of the head in the riser. Also,

$$q_t = A_t \frac{dh_t}{dt} = - q_a + F(h_g) \tag{4}$$

The quantities q_r and q_a are defined as

$$q_r = A_r \frac{dh_r}{dt} \tag{5}$$

$$q_a = C_r A_a [2g_r(h_t - h_g)]^n \tag{6}$$

where the equation for q_a is the standard equation for flow through an orifice. When the value of these quantities and the value of q_e as given by equation 1 are substituted in equations 3 and 4, there is obtained

$$q_c - q_g = (C_e + A_r) \frac{dh_r}{dt} - C_r A_a [2g_r(h_t - h_g)]^n + F(h_g) \tag{7}$$

$$A_t \frac{dh_t}{dt} = - C_r A_a [2g_r(h_t - h_g)]^n + F(h_g) \tag{8}$$

The three equations, 2, 7, and 8, are used in the computer studies.

Flow Through the Control Gate into a Turbine

The energy of a penstock system at the control gate is assumed to equal the sum of the steady state load of the turbine and the kinetic energy of the rotating parts. Thus

$$Kh_g q_g = l_d + n \frac{jdn}{dt} \tag{9}$$

The flow through the gate may be described as

$$q_g = C_g g (2g_r h_g)^n \tag{10}$$

Analysis of Penstock System with Simple Surge Tank for Governor Study

Equation 2 and the equation

$$(q_c - q_g) = (A_t + C_e) \frac{dh_g}{dt} \tag{11}$$

describe the penstock with a simple surge tank shown in Fig. 6. When the effect of the penstock is included in a governor study, small changes in speed and gate movement relative to steady state values permit simplifications in equations 2, 9, 10, and 11 describing a turbine-penstock system.

The quantity q_g may be eliminated from equations 9 and 11 by substituting its value as given by equation 10 into these two equations. Then

$$\frac{dh_g}{dt} = \frac{1}{(A_t + C_e)} \left[q_c - C_g g (2g_r h_g)^{\frac{1}{2}}\right] \tag{12}$$

$$KC_g g (2g_r)^{\frac{1}{2}} h_g^{\frac{3}{2}} = l_d + n \frac{jdn}{dt} \tag{13}$$

where the exponent n of equation 10 is assumed to be 1/2.

Certain of the steady state quantities are related as follows:

(from 2)

$$H_{go} = H_o' - K_c Q_{co}^2 \tag{14}$$

(from 12) $$Q_{co} = C_g G_o (2g_r H_{go})^{\frac{1}{2}} \tag{15}$$

(from 13) $$KC_g G_o (2g_r)^{\frac{1}{2}} H_{go}^{\frac{3}{2}} = L_{do} \tag{16}$$

When the value of H_{go} of 14 is substituted in the equation

$$h_g = H_{go} + H_g,$$

which defines h_g in terms of the steady state and fluctuating values, there is obtained

$$h_g = H_o' - K_c Q_{co}^2 + H_g \tag{17}$$

The following equation results when the instantaneous quantities of 2 are replaced by their steady state and fluctuating quantities, and when the value of h_g given by 17 is substituted:

$$- H_g = \frac{L_c}{A_c g_r} \frac{dQ_c}{dt} + K_c(2Q_{co}Q_c + Q_c^2) \tag{18}$$

In terms of the steady state and fluctuating quantities, equation 12 becomes

$$\frac{dH_g}{dt} = \frac{1}{(A_t+C_e)} \left[Q_{co} + Q_c - C_g(2g_r)^{\frac{1}{2}} G_o H_{go}^{\frac{1}{2}} (1 + \frac{G}{G_o})(1 + \frac{H_g}{H_{go}})^{\frac{1}{2}} \right]$$

or

$$\frac{dH_g}{dt} = \frac{1}{(A_t+C_e)} \left[Q_{co} + Q_c - Q_{co}(1 + \frac{G}{G_o})(1 + \frac{1}{2} \frac{H_g}{H_{go}} - \frac{1}{8} \frac{H_g^2}{H_{go}^2} + \text{---}) \right] \tag{19}$$

where $(1 + \frac{H_g}{H_{go}})^{1/2}$ is written in series form and $C_g(2g_r)^{1/2} G_o H_{go}^{1/2}$ is replaced by Q_{co} (equation 15).

By similar substitutions, equation 13 becomes

$$KC_g(2g_r)^{\frac{1}{2}} G_o H_{go}^{\frac{3}{2}} (1 + \frac{G}{G_o})(1 + \frac{H_g}{H_{go}})^{\frac{3}{2}} = (L_{do} + L_d) + (N_o + N) \frac{JdN}{dt}$$

or

$$L_{do}(1 + \frac{G}{G_o})(1 + \frac{3}{2} \frac{H_g}{H_{go}} + \frac{3}{8} \frac{H_g^2}{H_{go}^2} \text{----}) = (L_{do} + L_d) + (N_o + N) \frac{JdN}{dt} \tag{20}$$

where $(1 + \frac{H_g}{H_{go}})^{3/2}$ is written in series form and $KC_g(2g_r)^{1/2} G_o H_{go}^{3/2}$ is replaced by L_{do} (equation 16).

The following assumptions may be made if the fluctuating quantities are small relative to the steady state quantities:

1. All second order terms, or higher, of the fluctuating quantities may be neglected.
2. Products of unlike fluctuating quantities may be neglected.
3. $N_o = N_o + N$

When these assumptions are applied to equations 18, 19, and 20 there is obtained

$$- H_g = 2K_c Q_{co} (\frac{L_c}{2A_c g_r K_c Q_{co}} p + 1) Q_c \tag{21}$$

$$\frac{Q_{co}}{2H_{go}}\left[\frac{2H_{go}}{Q_{co}}(A_t + C_e)p + 1\right]H_g = Q_c - \frac{Q_{co}}{G_o}G \tag{22}$$

$$\frac{L_{do}}{G_o}G + \frac{3}{2}\frac{L_{do}}{H_{go}}H_g = L_d + N_o\frac{JdN}{dt} \tag{23}$$

If we let

$$\alpha = \frac{1}{2K_cQ_{co}} \tag{24}$$

$$\beta = \frac{2H_{go}}{Q_{co}} \tag{25}$$

$$T_c = \frac{L_c}{2A_cgrK_cQ_{co}} \tag{26}$$

$$T_{te} = \frac{2H_{go}}{Q_{co}}(A_t + C_e) \tag{27}$$

$$T_m = N_oJ\ , \tag{28}$$

equations 21, 22, and 23 becomes

$$-H_g = \frac{1}{\alpha}(T_cp + 1)Q_c \tag{29}$$

$$\frac{1}{\beta}(T_{te}p + 1)H_g = Q_c - \frac{Q_{co}}{G_o}G \tag{30}$$

$$\frac{L_{do}}{G_o}G + \frac{3}{2}\frac{L_{do}}{H_{go}}H_g = L_d + T_mpN \tag{31}$$

The Hydraulic Governor

The hydraulic governor used in the study is shown in Fig. 7. The following equations taken from Reference 1 describe the governor system:

Forward transfer from (R) to (C)

$$y = \frac{N}{K_4} \tag{32}$$

$$x_2 = \frac{a+b}{a}y - \frac{b}{a}x_1 \tag{33}$$

$$pG = f(K_3x_2) \tag{34}$$

Feedback transfer from (C) to (A)

$$\frac{x_1-G}{G-x_4} = \frac{d}{e} \tag{35}$$

$$K_1x_1d = eK_2px_4 \tag{36}$$

After substituting the value of y of 32 in 33, there is obtained

$$x_2 = \frac{a+b}{K_4a}N - \frac{b}{a}x_1 \tag{37}$$

When x_4 is eliminated from 35 and 36, we have

$$x_1 = \frac{\frac{K_2e}{K_1d}(e+d)pG}{1 + \frac{K_2e^2}{K_1d^2}p} \tag{38}$$

Equations 29, 30, 31, 34, 37, and 38 describe a complete penstock governor system if it is assumed

1. A single hydroelectric unit supplies the load.

2. The generator supplies real power with instantaneous voltage regulation so that the power is independent of speed and the torque varies inversely with speed.
3. The turbine efficiency is constant for small changes in speed, head and gate opening.

APPENDIX II

Explanation of the Computer Circuit for the Study of a Penstock with a Differential Surge Tank

After rearrangement of terms of equations 2, 7, and 8 of Appendix I, the system equations are the following:

$$\frac{L_c}{g_r A_c} \frac{dq_c}{dt} = h' - h_g - K|q_c| q_c \tag{39}$$

$$(C_e + A_r) \frac{dh_g}{dt} = q_c - q_g + C_r A_a [2g_r(h_t - h_g)]^n - F(h_g) \tag{40}$$

$$- A_t \frac{dh_t}{dt} = C_r A_a [2g_r(h_t - h_g)]^n - F(h_g) \tag{41}$$

From these equations, the computer may be connected as shown in Fig. 3. The water may reach the main outer tank by flowing through the riser port of area A_a or may, upon large load changes, overflow the riser. The flow through the port is an exponential function of the pressure across it. Hence a function generator is necessary in the computer setup. Also the overflow, $F(h_g)$, is simulated in the following manner with reference to Fig. 3: A limiter is set to limit the riser water level at the riser height. Then the difference between the limited and unlimited riser level is taken, differentiated, multiplied by the riser area A_r, and multiplied by a suitable time lag to account for the time for the overflow to fall to the outer tank level. Thus, the generation of $F(h_g)$ is accomplished in the limiter, amplifier 9, and the potentiometer that follows.

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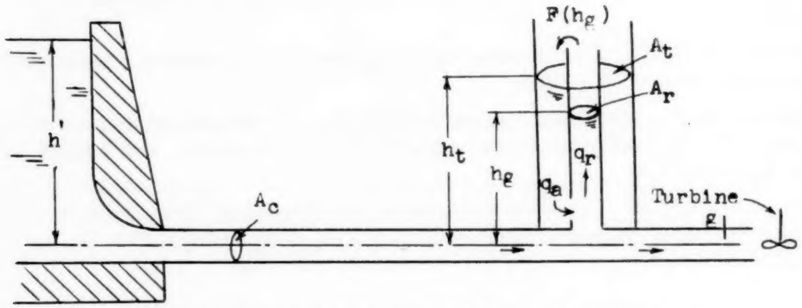


Figure 1. Penstock System with a Differential Surge Tank

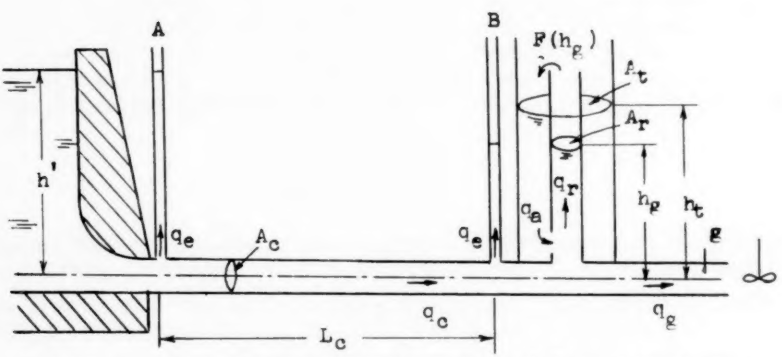


Figure 2. Equivalent Penstock System of Figure 1 where the Effect of Elastic Walls is Simulated in a Lumped Parameter Sense by Surge Tanks A and B on the Ends of the Conduit

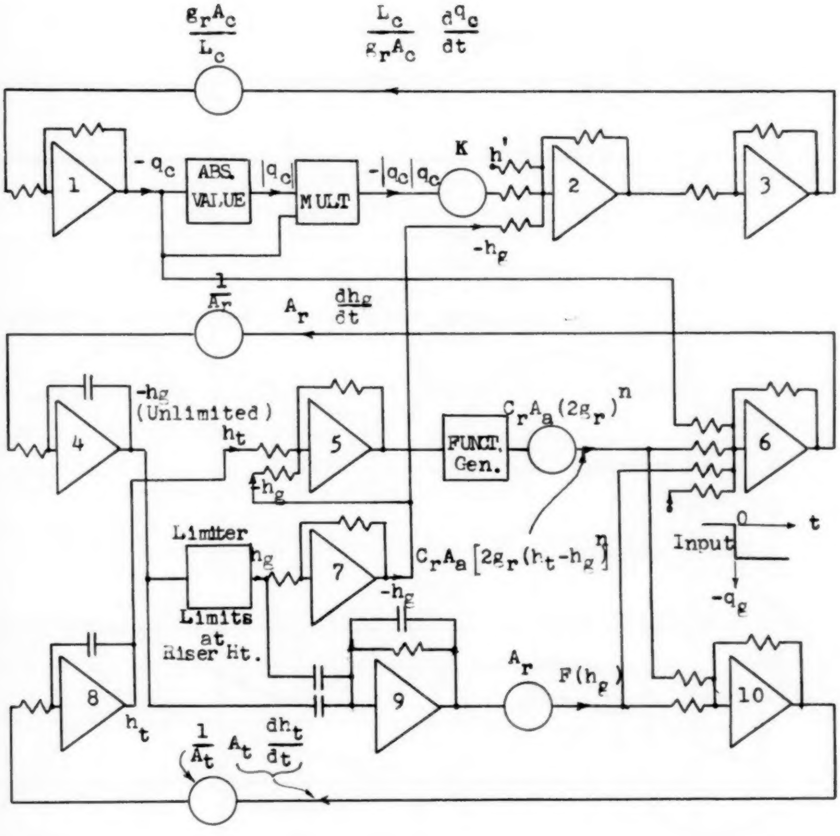


Figure 3. Computer Circuit Diagram for Study of Penstock With Differential Surge Tank

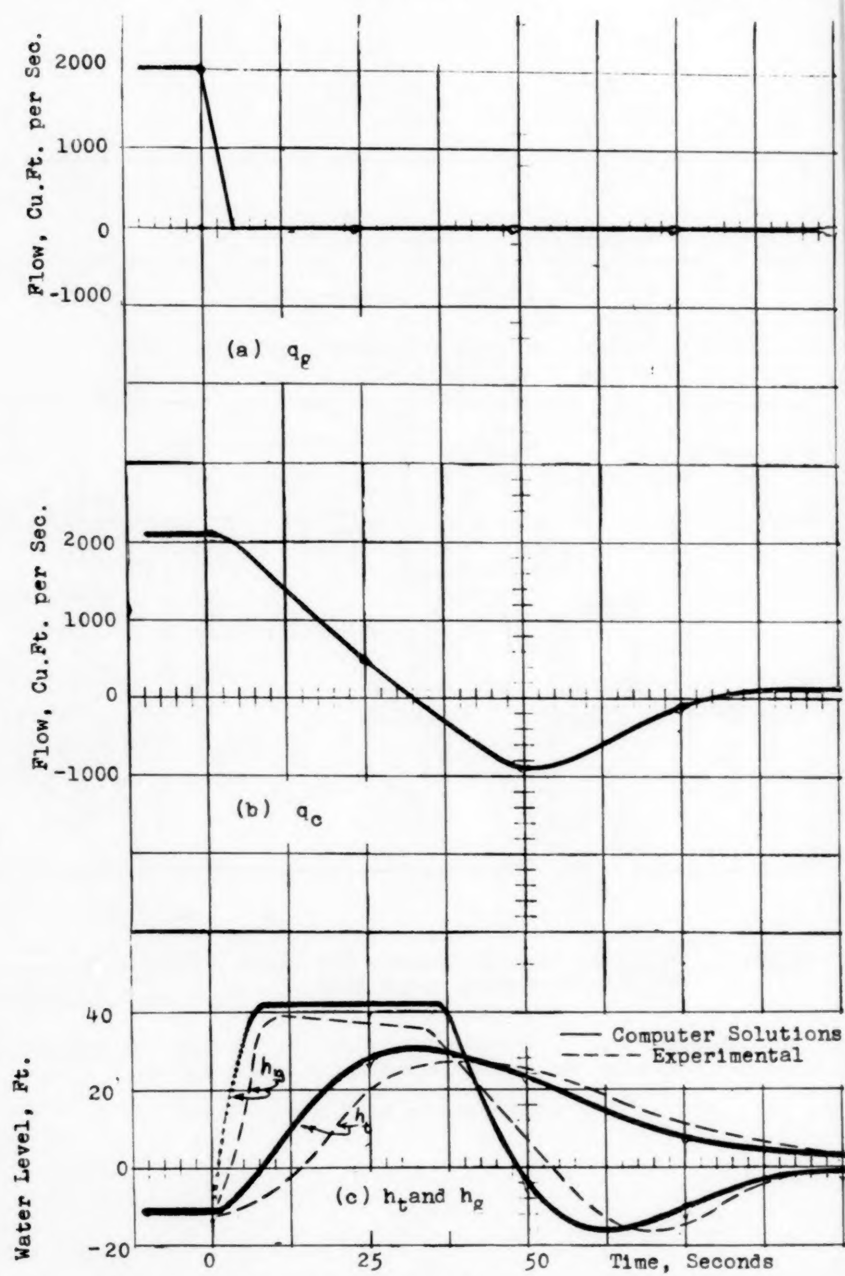
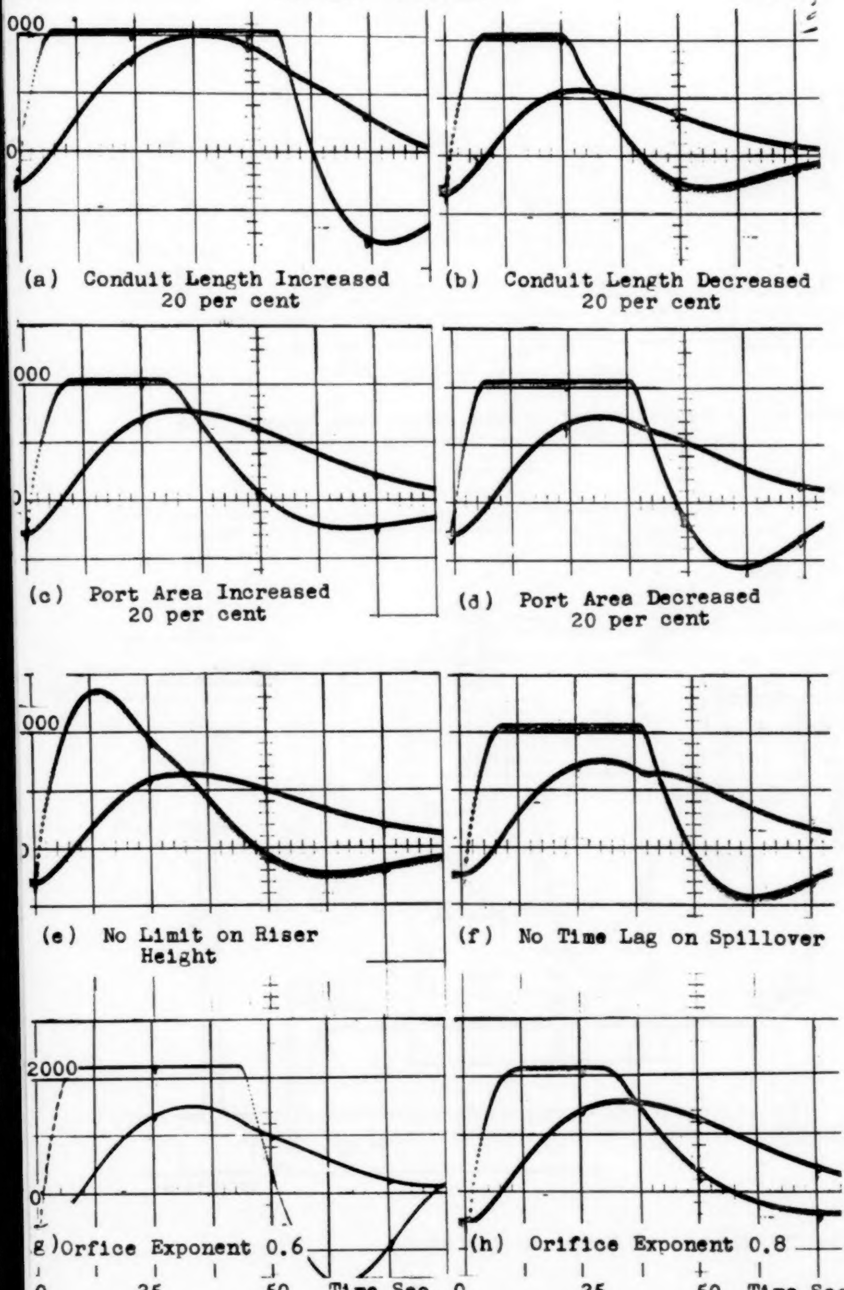


Figure 4 Computer Solution to Problem of Penstock With Differential Surge Tank



(a) Conduit Length Increased 20 per cent

(b) Conduit Length Decreased 20 per cent

(c) Port Area Increased 20 per cent

(d) Port Area Decreased 20 per cent

(e) No Limit on Riser Height

(f) No Time Lag on Spillover

(g) Orfice Exponent 0.6

(h) Orifice Exponent 0.8

Figure 5. Computer Solutions for h_t and h_g of Penstock with Differential Surge Tank for Different Values of Parameters

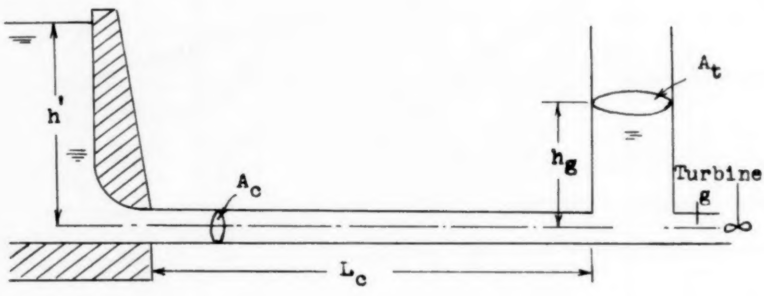


Figure 6. A Penstock and Simple Surge Tank System Used in the Study of the Governor of Figure 7

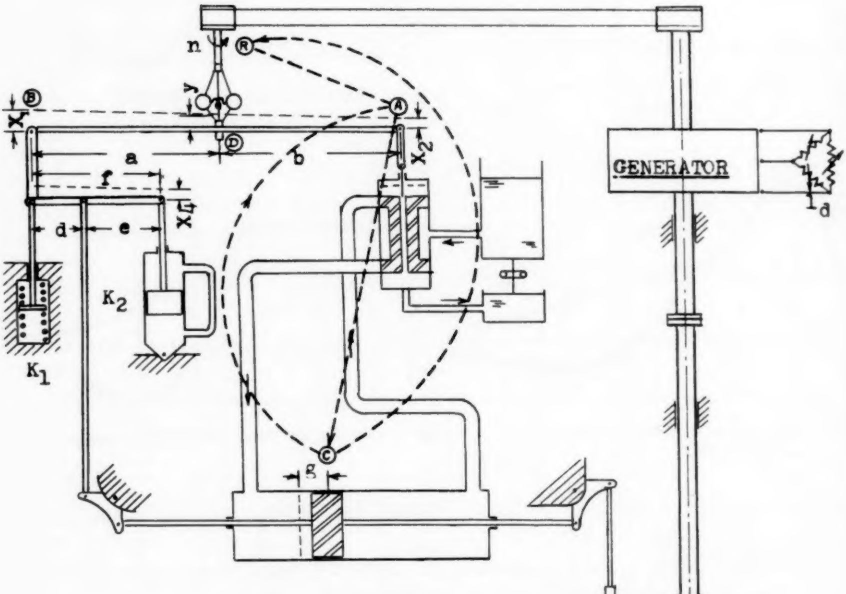


Figure 7. Sketch of Governor for Study

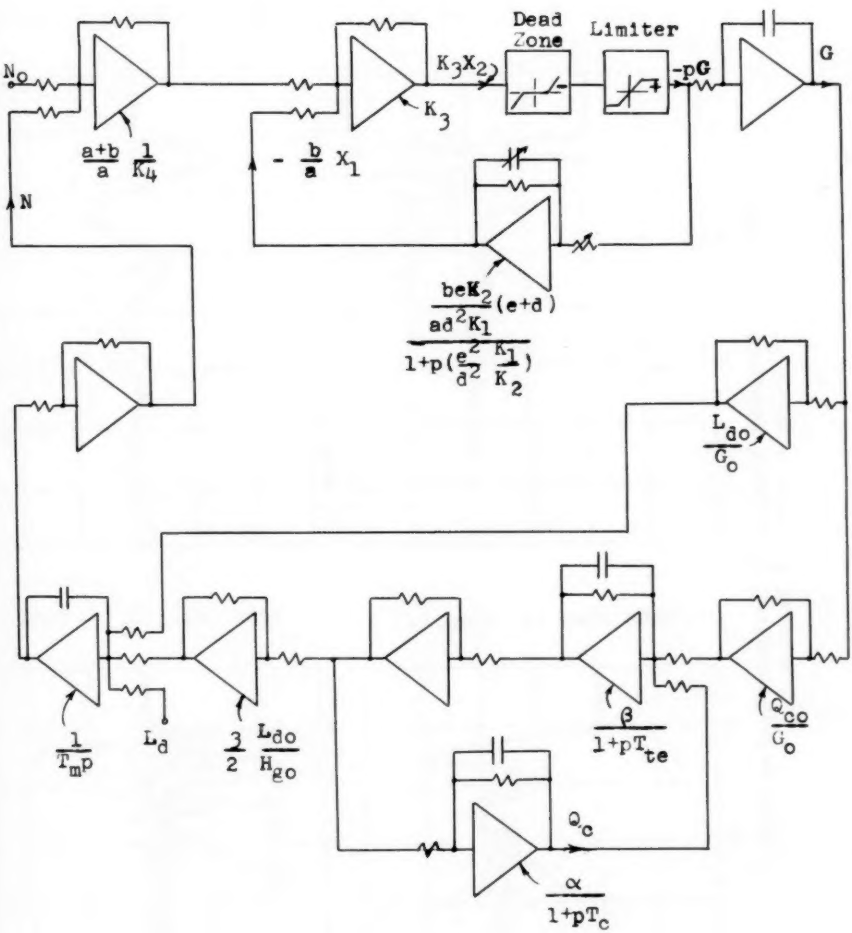


Figure 8. Computer Circuit of Governor System (Fig. 7) Including Penstock with Simple Surge Tank (Fig.6)

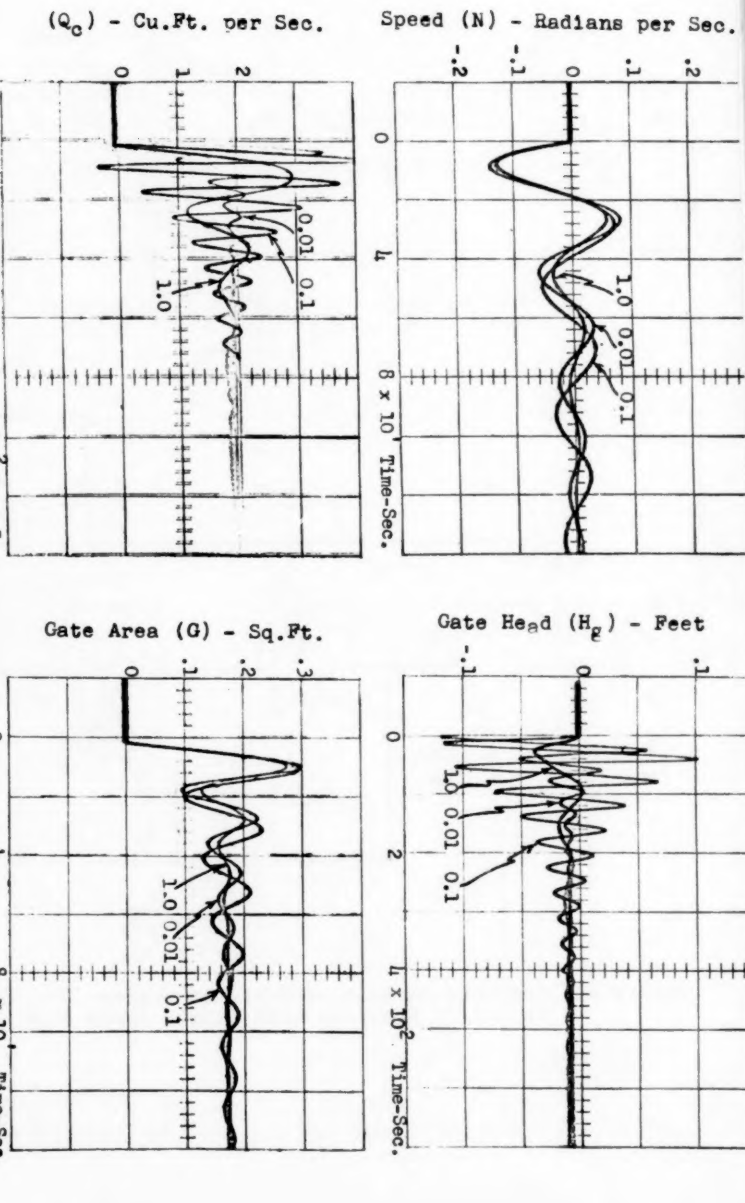


Figure 9. Computer Results of Governor System Study Showing Effects of Varying T_{te}

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1113 is identified as 1113 (HY6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1956.

VOLUME 82 (1956)

DECEMBER: 1113(HY6), 1114(HY6), 1115(SA6), 1116(SA6), 1117(SU3), 1118(SU3), 1119(WW5), 1120(WW5), 1121(WW5), 1122(WW5), 1123(WW5), 1124(WW5)[c], 1125(BD1)[c], 1126(SA6), 1127 (SA6), 1128(WW5), 1129(SA6)[c], 1130(PO6)[c], 1131(HY6)[c], 1132(PO6), 1133(PO6), 1134(PO6), 1135(BD1).

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JANUARY: 1136(CP1), 1137(CP1), 1138(EM1), 1139(EM1), 1140(EM1), 1141(EM1), 1142(SM1), 1143(SM1), 1144(SM1), 1145(SM1), 1146(ST1), 1147(ST1), 1148(ST1), 1149(ST1), 1150(ST1), 1151(ST1), 1152(CP1)[c], 1153(HW1), 1154(EM1)[c], 1155(SM1)[c], 1156(ST1)[c], 1157(EM1), 1158 (EM1), 1159(SM1), 1160(SM1), 1161(SM1).

FEBRUARY: 1162(HY1), 1163(HY1), 1164(HY1), 1165(HY1), 1166(HY1), 1167(HY1), 1168(SA1), 1169(SA1), 1170(SA1), 1171(SA1), 1172(SA1), 1173(SA1), 1174(SA1), 1175(SA1), 1176(SA1), 1177(HY1)[c], 1178(SA1), 1179(SA1), 1180(SA1), 1181(SA1), 1182(PO1), 1183(PO1), 1184(PO1), 1185(PO1)[c].

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OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4), 1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395(HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404(HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(EM4), 1413(EM4), 1414(PO5), 1415(EM4)[c], 1416(PO5)[c], 1417 (HY5)[c], 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)[c], 1423(SA5), 1424(EM4), 1425(CP2).

NOVEMBER: 1426(SM4), 1427(SM4), 1428(SM4), 1429(SM4), 1430(SM4)[c], 1431(ST6), 1432(ST6), 1433(ST6), 1434(ST6), 1435(ST6), 1436(ST6), 1437(ST6), 1438(SM4), 1439(SM4), 1440(ST6), 1441(ST6), 1442(ST6)[c], 1443(SU2), 1444(SU2), 1445(SU2), 1446(SU2), 1447(SU2), 1448(SU2)[c].

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c. Discussion of several papers, grouped by Divisions.

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